UNIVERSIDAD NACIONAL DE INGENIERÍA

FACULTAD DE INGENIERÍA CIVIL

ESTUDIO DEL COMPORTAMIENTO DEL CONCRETO DE MEDIANA A BAJA RESISTENCIA, CON LA INCORPORACIÓN DE FIBRAS DE ACERO Y CEMENTO PÓRTLAND TIPO I ANDINO

TESIS

Para optar el Título Profesional de:

INGENIERO CIVIL

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Lima – Perú

2001

EN MEMORIA :

De Ángel, mi Padre quien guía mis pasos y de quien estoy seguro, hubiese estado muy orgulloso por la tarea cumplida

DEDICATORIA :

A María, mi Madre como muestra de mi eterno agradecimiento por su amor y fortaleza; a quien admiro por su dedicación y a quien le debo lo que soy.

A mis Hermanos: Antonio, Gladis, Efraín, Jhon, Wilmer y Marina, quienes siempre me apoyaron e incentivaron a que este proyecto se haga realidad.

A todos mis amigos y a una persona muy especial que siempre me ha apoyado y está conmigo N.R.A.S

AGRADECIMIENTO :

Al Ing. Carlos Barzola Castelú, por su apoyo desinteresado y que me llevo a conseguir la culminación de esta tesis.

A la empresa Insomin, Al Ing. Carlos Alania por todo el apoyo brindado.

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INTRODUCCIÓN

En la actualidad, se plantea un reto muy importante en el desarrollo de la tecnología del concreto, el avance y el perfeccionamiento exige una investigación continua de la calidad del mismo, determinando pues su comportamiento y/o características tanto en el estado fresco como en el estado endurecido. Como puede verse, la proporción y tipos de los ingredientes establecen en parte la calidad del concreto y por lo tanto la calidad del sistema estructural total. Así mismo las obras de gran envergadura como proyectos de construcción civil, subterránea y minera requieren contar con técnicas apropiadas para su desarrollo.

Controlar la calidad de un producto consiste en evaluar ciertos criterios y parámetros técnicos antes, durante y después del proceso productivo, para garantizar que el resultado final satisfaga los requerimientos esperados.

Si el concreto es sometido a cargas de esfuerzos de Tensión y cargas de Impacto, este se torna muy frágil y debido a esto es necesario reforzar el concreto con varillas de refuerzo de acero o mallas. Entre los insumos que se viene considerando para mejorar la falta de ductilidad es el uso de Fibras de Acero.

Los principales efectos que trae consigo la incorporación de fibras de acero al concreto podemos resumirlos en lo siguiente: Mejora el comportamiento a flexo tracción, incremento de la resistencia a la rotura, aumento de la resistencia a la tracción, fuerte incremento en la resistencia al impacto y choque.

El empleo de fibras para mejorar la isotropía de un material no es algo desconocido, así tenemos el adobe de barro reforzado con paja. Al mismo concreto armado podríamos considerarlo, en el límite, como un concreto con fibras gruesas orientadas.

Las fibras empleadas en el concreto reforzado son discontinuas, presentando una distribución discreta y uniforme. La efectividad de la acción reforzante y la eficacia en la transmisión de tensiones depende de muchos factores pero, especialmente, de la naturaleza y del tipo de fibra empleado.

Para nuestro tema de investigación se ha utilizado Fibra de Acero Nacional INSONEX, de la compañía INSOMIN, diseño de mezclas para las relaciones agua/cemento: 0.60, 0.65, 0.70. empleando proporciones de 35, 45, 55 kg/m^3 de concreto,

Las características de la fibra de acero son las siguientes: Geometría Ondulada, Longitud de 40 mm, Diámetro de 0.80 mm, Longitud de la onda de 5 mm, Forma de suministro en cajas de 40 kg.

Es de esperar que este trabajo de investigación sea de utilidad a los profesionales que se relacionan diariamente con la tecnología del concreto y utilizarla de acuerdo a la necesidad y los requerimientos que estos exijan como puede ser las condiciones ambientales y climatológicas. De esta manera se estará en capacidad de obtener, pues, un concreto eficiente y deseable que satisfaga los requisitos de resistencia y condiciones de servicio del diseñador. Se desea así, crear una mayor expectativa en el tema y de este modo se continúe realizando más investigaciones al respecto.

CAPÍTULO 1

CEMENTO PÓRTLAND

1.1 DEFINICIÓN

CEMENTO

Es un material pulverizado que se denomina hidráulico porque fragua y endurece al reaccionar con el agua, formando una pasta capaz de endurecer tanto bajo el agua y el aire. La patente del primer cemento Pórtland se le acredita a Joseph Aspdin en 1824, quien los fabrica por un proceso similar al actual, llamando a su producto cemento Pórtland.

CEMENTO PÓRTLAND

Es un aglomerante hidrófilo producido artificialmente por la pulverización del CLINKER, con la adición de 5% en peso de yeso natural (sulfato de calcio); que al combinarse con el agua produce una masa capaz de endurecer como la piedra, el fenómeno químico es conocido como hidratación, cuya velocidad de reacción esta directamente influenciada por la finura del cemento e inversamente proporcional al tiempo, por lo que inicialmente es muy rápido y va disminuyendo paulatinamente, el proceso es exotérmico por que genera calor hacia el exterior denominado calor de hidratación. A continuación se menciona algunas fechas de cómo a evolucionado el Cemento Pórtland .

1824: - James Parker, Joseph Aspdin patentan al Cemento Pórtland, materia que obtuvieron de la calcinación de alta temperatura de una Caliza Arcillosa.

1845: - Isaac Johnson obtiene el prototipo del cemento moderno quemado, alta temperatura, una mezcla de caliza y arcilla hasta la formación del "clinker".

1868: - Se realiza el primer embarque de cemento Pórtland de Inglaterra a los Estados Unidos.

1871: - La compañía Coplay Cement produce el primer cemento Pórtland en lo Estados Unidos.

1904: -La American Standard For Testing Materials (ASTM), publica por primera ves sus estándares de calidad para el cemento Pórtland.

1906: - Se instala la primera fabrica para la producción de cemento en México, con una capacidad de 20,000 toneladas por año.

Se asume que el cemento que se expende en el mercado, cumple con los requisitos normativos y en general es así, por lo que no se realizan verificaciones de su calidad. Sin embargo debe observarse si se presenta algún comportamiento anormal para tomar las previsiones del caso.

HIDRÓFILO

Materia que tiene la propiedad de absorber agua.

CLINKER

El Clinker del cemento Pórtland se obtiene por la calcinación a elevada temperatura (1400°C - 1450°C) hasta la fusión parcial (clinkerización) de una mezcla convenientemente proporcionada y homogeneizada de materiales debidamente seleccionados. Las materias primas más importantes son las calizas y arcillas.

Ahora si el Clinker fuera molido finamente para ser utilizado como cemento, en el momento de su mezcla con el agua fraguaría casi de inmediato, no permitiendo tanto su manipuleo como su colocación.

Es por esta razón, que en el momento de su molienda se le adiciona sulfato de calcio (yeso), con el objeto de retardar el tiempo de fraguado.

El clinker se compone de la siguiente manera:

Silicato Tricálcico	: $3CaO.SiO_2$	(40% - 65%) = C_3S
Silicato Bicálcico	: $2CaO.SiO_2$	(10% - 30%) = C_2S
Aluminato Tricálcico	: $3CaO.Al_2O_3$	(7% - 15%) = C_3A
Aluminato Ferrato	: $4CaO.Al_2O_3FeO_3$	(4% - 15%) = C_4AF

1.2 TIPOS DE CEMENTOS

El cemento Pórtland normal se clasifica en cinco tipos diferentes, de acuerdo a las propiedades relativas de los compuestos principales y a las condiciones de uso; de acuerdo a las normas técnicas peruanas NTP y a las internacionales A.S.T.M., los cementos se clasifican en dos grandes grupos:

a) CEMENTOS PÓRTLAND NORMALES:

TIPO I. - De uso general, donde no se requieren propiedades especiales.

TIPO II. - De moderado resistencia a los sulfatos y moderado calor de hidratación. Para emplearse en estructuras con ambientes agresivos y/o en vaciados masivos.

TIPO III. - Desarrollo rápido de resistencia con elevado calor de hidratación. Para uso en clima fríos o en los casos en que se necesita adelantar la puesta en servicio de las estructuras.

TIPO IV. - De bajo calor de hidratación. Para concreto masivo.

TIPO V. - Alta resistencia a los sulfatos. Para ambientes muy agresivos.

En la actualidad se fabrican en el Perú los cementos Tipo I, Tipo II y Tipo V.

En el presente tema de investigación se utilizó el Cemento Pórtland Tipo I Andino

b) CEMENTOS PÓRTLAND ADICIONADOS:

Son cementos hidráulicos, que consisten de una mezcla intima y uniforme producida por la molienda conjunta del CLINKER con los materiales de adición y yeso, o por la mezcla separada del cemento Pórtland con dichas adiciones.

Independientemente a su forma de obtención, existe una gran variedad de tipos que deriva tanto de la clase del material incorporado, que puedan ser variados, como el porcentaje en que se encuentra la adición y de la presencia de aire incorporado principalmente.

Estos cementos, en el mundo, están reemplazando cada vez con mayor intensidad a los cementos Pórtland normales, debido no solamente a sus mejores características, sino porque también son una solución al alto consumo energético que se emplea en la fabricación del CLINKER y, en el caso de los cementos adicionados con escoria, se aprovecha este subproducto tradicionalmente desechado.

Es interesante destacar los cementos denominados “mezclados o adicionados” dado que algunos de ellos se usan en nuestro medio:

Tipo IS.- Cemento al que se ha añadido entre un 25% a 70% de escoria de altos hornos referido al peso total.

Tipo ISM.- Cemento al que se ha añadido menos de 25% de escoria de altos hornos referido al peso total.

Tipo IP.- Cemento al que se le ha añadido puzolana en un porcentaje que oscila entre el 15% y 40% del peso total.

Tipo IPM.- Cemento al que se le ha añadido puzolana en un porcentaje hasta del 15% del peso total.

c) CEMENTOS PÓRTLAND ADICIONADOS (ESPECIFICACIÓN DE LA PERFORMANCE) N.T.P. 334.082.

Esta nueva Norma Técnica Peruana 334.082, fue publicada el 24 – 07 – 1998. Los tipos de cemento Pórtland que cubren esta especificación, están clasificados de acuerdo a sus propiedades y son:

TIPO GU. - Cemento Pórtland adicionado para construcciones generales.
TIPO HE. - De alta resistencia inicial.
TIPO MS. - De moderado resistencia a los sulfatos.
TIPO HS.- De alta resistencia a los sulfatos.
TIPO MH. - De moderado calor de hidratación.
TIPO LH. - De bajo calor de hidratación.

Cuando el tipo no esta especificado, se aplicarán los requisitos del TIPO GU.

1.3 CARACTERÍSTICAS

1.3.1 COMPOSICIÓN QUÍMICA

Las características y propiedades del cemento están íntimamente ligadas a sus componentes y compuestos químicos. Se entiende como componentes a los minerales u óxidos aportados pos la materia prima, reaccionan entre si en el horno y forman productos más complejos; denominados compuestos primarios y secundarios; estos fueron establecidos por primera vez por Le Chatelier en el año 1852 y son los que definen el comportamiento del cemento hidratado.

1.3.1.1 COMPONENTES PRINCIPALES

Entre los componentes principales del cemento Pórtland tenemos:

LA CAL (CaO) : La cal u oxido de calcio representa un 61% a 67% del cemento. Proviene de la roca Caliza; luego de calentarla a una temperatura de 1000°C se descomponen oxido de calcio y anhídrido carbónico. El exceso de cal ocasiona inconsistencia y desintegración del cemento después del fraguado. Un contenido alto de cal pero no lo suficiente para considerarse excesivo, tiende a retardar el fraguado, pero produce una resistencia inicial alta. Muy poca cal puede producir cementos débiles.

LA SÍLICE (SiO_2) : La sílice u oxido de sílice forma alrededor de 17% a 25% en el cemento. Proviene en mayor proporción de la arenisca, cuarcita, arena de cuarzo etc. Es resistente e insoluble en el agua. Un contenido alto de sílice produce cemento de alta resistencia, de fraguado lento y mejora la resistencia contra el ataque químico.

LA ALUMINA (Al_2O_3) : La alumina u oxido de aluminio forma alrededor de 4 a 8% en el cemento. Proviene de la arcilla. Un alto contenido de alumina y bajo de sílice, produce un cemento de fraguado rápido y también de alta resistencia.

OXIDO FERRICO (Fe_2O_3) : Se encuentra en un 0.5 a 5%. El color gris en el cemento se debe a este oxido, el cual actúa en la misma forma que la elimina. Si el cemento es de color blanco este oxido no esta presente.

1.3.1.2 COMPONENTES SECUNDARIOS

PERDIDA POR IGNICIÓN : Es la disminución de peso de una muestra de cemento que fue calentada al rojo vivo (de 900°C a 1000°C) hasta obtener un peso constante. Según las normas NTP, la perdida por ignición para los cementos Pórtland tipo I, II, y V debe ser de 3% como máximo; si se supera este valor el cemento no podrá ser utilizado en elementos estructurales, debido a que el cemento podría estar en estado de prehidratación o carbonatación que puede ser producido durante el proceso de fabricación, o también por un almacenamiento incorrecto y prolongado.

RESIDUO INSOLUBLE : Nos muestra que parte de la porción arcillosa no se a combinado y no es soluble. Además de indicar el nivel de perfección que se da en el horno, durante la cocción. Si consideramos la mezcla cruda del cemento vemos que la parte arcillosa, durante la cocción reacciona con la cual transformándose en minerales del klinker con solubilidad en los ácidos. Sin embargo, siempre existe una porción de cemento que no ha logrado disolverse con ácido clorhídrico, a esta porción se le conoce como Residuo insoluble.

ANHÍDRIDO SULFURICO (SO_3) : Presente en pequeñas cantidades; proviene del yeso que se le añade al klinker para retardar la fragua, permite realizar el calculo del valor de calcio presente en el cemento, así como la cal combinada y tan bien el contenido de azufre, limitándose al 2.5 o 3%.

1.3.1.3 COMPUESTOS PRINCIPALES

Constituyen de un 90 a 95% del cemento Pórtland.

SILICATO TRICALCICO ($3CaO.SiO_2 = C_3S$) : También conocido como Alita. Este compuesto es el factor principal del fraguado inicial y del rápido endurecimiento; genera un alto calor de hidratación. La cantidad formada en la reacción de fraguado tiene un marcado efecto sobre la resistencia del concreto en sus primeras etapas, principalmente en los primeros 14 días. Deberá limitarse el contenido de C_3S en los cementos para obras de grandes masas de concreto, no debiendo rebasarse un 35%, con objeto de evitar valores elevados de calor de hidratación, en tales casos se preferirá contenidos altos en silicatos bicalcicos.

SILICATO BICALCICO ($2CaO.SiO_2 = C_2S$) : También conocido como Belita . La formación de este compuesto se desarrolla lentamente con un grado lento de evolución de calor. Es principalmente responsable del incremento progresivo de la resistencia, lo cual ocurre a los 14 a 28 días y en adelante, tienes una resistencia alta al ataque químico y también un encogimiento por secado relativamente bajo.

ALUMINATO TRICALCICO ($3CaO.Al_2O_3 = C_3A$) : Libera una gran cantidad de calor durante los primeros días de endurecimiento. También contribuye ligeramente a la resistencia temprana. Los cementos con bajos porcentajes de este compuesto especialmente resistentes a los suelos y aguas que tengan sulfatos. Deberá usarse cementos con C_3A en proporción no mayor al 5% cuando el sulfato, soluble en agua en el suelo excede del 0.2%, o en el agua freática excede de 1000 partes por millón.

FERRO-ALUMINATO TETRACALCICO ($4CaO.Al_2O_3$ Fe_2O_3 = C_4AF) : Conocido como Celita. No participa prácticamente sobre la resistencia mecánica u otras propiedades del cemento endurecido. Se hidrata con relativa rapidez. Tiene un pequeño calor de hidratación en gran velocidad de fraguado.

1.3.1.4 COMPUESTOS SECUNDARIOS

Constituyen un 5 a 10% del cemento Pórtland.

OXIDO DE MAGNESIO (MgO) : Proviene de la piedra caliza, roca calcárea y escorias. La presencia este óxido le da al cemento un color verde grisáceo. Actúa como fundente en la formación del klinker y aporta fase liquida. En contacto con el agua se hidrata y aumenta de volumen. La expansión se manifiesta lenta en concreto ya fraguados y endurecidos. Según las normas NTP, los cementos Pórtland tipo I, no deben tener más de 5% de oxido de magnesio; y 6% como máximo para el resto de tipos de cementos.

ÓXIDO DE SODIO ÓXIDO DE POTASIO (Na_2O y K_3O) : Se le conoce con el nombre de álcalis, se eliminan normalmente con los gases producidos en la calcinación del cemento, se encuentra presentes en el producto terminado solamente en pequeñas cantidades. Si por alguna razón se encuentra en cantidades excesivas, causaran eflorescencia.

En el cuadro N° 1.1 se muestra el análisis químico del cemento Pórtland tipo I Andino.

CUADRO N° 1.1
ANÁLISIS QUIMICO DEL CEMENTO PÓRTLAND TIPO I ANDINO

COMPUESTO	SIMBOLO	%	NORMA ITINTEC 333.009
Oxido de Calcio	CaO	64.18	-
Oxido de Silicio	SiO_2	21.86	-
Oxido de Aluminio	Al_2O_3	4.81	-
Oxido de Fierro	Fe_2O_3	3.23	-
Oxido de Potasio	K_2O	0.65	-
Oxido de Sodio	Na_2O	0.15	-
Oxido de Azufre	SO_3	2.41	max 3.5%
Oxido de Magnesio	MgO	0.96	max 5.0%
Silicato Tricalcico	C_3S	51.33	-
Silicato Bicalcico	C_2S	23.95	-
Aluminato Tricalcico	C_3A	7.28	-
Ferroaluminato Tetracalcico	C_4AF	9.82	-
Cal Libre	-	0.59	-
Perdida por Ignición	P.I	1.24	max 3.0%
Residuos Solubles	R.I	0.42	max 1.0%

1.4 PROPIEDADES FÍSICAS

Las principales características físicas del cemento Pórtland Tipo I Andino son las siguientes:

1.4.1 PESO ESPECÍFICO N.T.P 334.005

El peso específico del cemento corresponde al de un material compactado y su valor suele variar entre 3.0 gr/cm^3 y 3.2 gr/cm^3. La norma norteamericana considera un valor promedio de 3.15, este valor corresponde a un cemento Pórtland normal;. Por lo que estos valores servirán como parámetros de aproximación. Esta es la única propiedad del cemento que se emplea directamente en el computo de las proporciones de la mezcla de concreto.

El método de ensayo para determinar el peso específico de los cementos es por medio de un frasco volumétrico de Lechatelier.

El peso específico del cemento Pórtland Tipo I Andino es 3.12 gr/cm^3

1.4.2 CONSISTENCIA NORMAL NTP 334.002

La cantidad de agua que se requiere para una pasta de consistencia normal; se expresa como porcentaje en peso del cemento utilizado. El conocimiento de esta propiedad es la base para la determinación del tiempo de fraguado de los cementos.

La consistencia normal para el cemento Pórtland Tipo I Andino es 22.15%

1.4.3 TIEMPO DE FRAGUADO N.T.P. 334.006

Cuando el cemento se mezcla con agua, las reacciones químicas que se producen originan cambios en la estructura de la pasta, conservando la mezcla su plasticidad durante un cierto tiempo, desde poco minutos hasta varias horas, para luego ocurrir varios fenómenos sucesivos.

FRAGUA INICIAL

Condición de la pasta de cemento en que se aceleran las reacciones químicas, empieza el endurecimiento y la perdida de la plasticidad, midiendo en términos de la resistencia a deformarse, es la etapa en que se evidencia el proceso exotérmico donde se genera el ya mencionado calor de hidratación, que es consecuencia de las reacciones químicas.

En esta etapa la pasta puede remezclarse sin producirse deformaciones permanentes ni alteraciones en la estructura que aún esta en formación.

La Fragua Inicial para el cemento Pórtland Tipo I Andino esta comprendida entre 1:58 a 2:24

FRAGUADO FINAL

Se obtiene al termino de la etapa del fraguado inicial, caracterizándose por endurecimiento significativo y deformaciones permanentes. La estructura del gel está constituida por ensamblaje definitivo de sus partículas endurecidas.

Se dice que la pasta de cemento a fraguado cuando logra una rigidez suficiente como para soportar una presión determinada de tipo arbitrario, ejercida por agujas pertenecientes a los aparatos de GILLMORE y VICAT.

La Fragua Final para el cemento Pórtland Tipo I Andino esta comprendida entre 3:08 a 3:45

FALSO FRAGUADO

El fenómeno de falso fraguado se manifiesta durante o después del amasado y se caracteriza por un brusco aumento de la viscosidad de la pasta sin gran desprendimiento de calor. Un amasado adicional vuelve dar a la pasta su plasticidad inicial, sin que las resistencias finales se modifiquen, no debe añadirse agua, el falso fraguado proviene de la deshidratación del yeso durante la molienda conjunta con el Clinker.

La deshidratación depende del tiempo de molido, del tanto por ciento de humedad y de la temperatura. Durante el amasado, el semi hidratado, muy ávido de agua, forma cristales de yeso los cuales dan rigidez a la pasta.

1.4.4 FINURA N.T.P. 334.002

La finura del cemento afecta la rapidez de la hidratación. Al aumentar la finura del cemento aumenta la rapidez a la que se hidrata el cemento, acelerando la adquisición de resistencia; observando que el agua necesaria para obtener un concreto con cierto revenimiento disminuye. Los efectos del aumento de finura en la resistencia se manifiestan principalmente durante los primeros 7 días.

La finura para el cemento Pórtland Tipo I Andino esta comprendida entre 3210 cm^2/gr a $3340 cm^2/gr$.

1.4.5 CALOR DE HIDRATACIÓN N.T.P. 334.064

La fragua y el endurecimiento de la pasta de cemento son producto de las reacciones dadas entre los componentes del cemento y el agua. Estas reacciones general una cantidad de calor conocido con el nombre de calor de hidratación que depende de la composición química y de la fineza del cemento. En lo referente a la composición química, la cal es el compuesto que ejerce mayor influencia.

Se sabe que, en cuanto a su fineza, un incremento de esta produce un mayor calor de hidratación. Asimismo una alta temperatura inicial de curado, acelera el desarrollo del calor de hidratación. La utilidad del conocimiento de esta propiedad física radica en que, en base a ella, se puede determinar que tipo de cemento usar en la construcción de determinadas obras. El calor de hidratación cumple funciones de auto protección cuando se trata de un medio de clima frío.

El calor de hidratación para el cemento Pórtland Tipo I Andino es: a los 07 días 64.93 cal/gr.

1.5 CARACTERÍSTICAS MECÁNICAS

1.5.1 RESISTENCIA A LA COMPRESIÓN Y TRACCIÓN N.T.P. 334.051

Se define a la capacidad del cemento para soportar esfuerzos sin falla. La velocidad de desarrollo de la resistencia es mayor durante el período inicial de endurecimiento, haciéndose más lenta a través del tiempo. El valor de la resistencia a los 28 días se considera como la resistencia del cemento. Las resistencias mecánicas del cemento están en función de la finura, de la composición química, del grado de hidratación y el contenido de agua en la pasta.

Edad	Resistencia a la Compresión (Kg/cm^2)
3	197
7	260
28	340

1.5.2 ESTABILIDAD DE VOLUMEN N.T.P. 334.054

La determinación de esta propiedad nos permite obtener las variaciones volumétricas que tienen lugar en la pasta de cemento cuando es sometida a vapor saturado y a una presión determinada. La determinación de estas variaciones nos indica la capacidad de cambio de volumen de los elementos estructurales previéndose entonces la posibilidad de agrietamientos o cuando estos cambios son importantes.

La estabilidad de volumen para el cemento Pórtland Tipo I Andino es 0.07%

CUADRO Nº 1.2
PROPIEDADES FÍSICAS DEL CEMENTO PÓRTLAND TIPO I ANDINO

PROPIEDADES FISICAS MECANICAS	UNIDAD	VALOR DE ENSAYO	LIMITES ASTM C150
Peso Específico	gr/cm^3	3.12	
Consistencia Normal	%	22.15	
Tiempo de Fraguado			
Fragua Inicial	h:m	1:58 - 2:24	Min 0:45
Fragua Final	h:m	3:08 - 3:45	Max 6:45
Superficie Especifica	cm^2/gr	3210 - 3340	Min 2800
Calor de Hidratación	cal/gr	64.93	
Resistencia a la Compresión			
03 Días	kg/cm^2	195 - 200	Min 122
07 Días	kg/cm^2	250 - 270	Min 2194
28 Días	kg/cm^2	340	Min 280
Estabilidad de Volumen	%	0.00 - 0.07	

CAPÍTULO 2

AGREGADOS

2.1 AGREGADO FINO

2.1.1 DEFINICIÓN

Es el proveniente de la desintegración natural o artificial (arena), que pasa el tamiz 3/8" (9.52 mm) y que cumple los limites establecidos en la Norma.

Se puede utilizar arena de molino o natural. La arena debe ser partícula dura, densa y durable de roca no recubierta y de contenido de humedad uniforme y estable; debe estar de acuerdo a la Norma ASTM C-33 actual.

CANTERA.- El agregado utilizado es el proveniente de la Cantera "GLORIA"

2.1.2 CARACTERÍSTICAS FÍSICAS

2.1.2.1 PESO ESPECÍFICO NTP 400.021

Es la relación a una temperatura estable, de la masa de un volumen unitario del material, a la masa del mismo volumen de agua destilada, libre de gas. Además el peso especifico es un indicador de la calidad en cuanto que los valores elevados corresponden a materiales de buen comportamiento, mientras que el peso especifico bajo generalmente corresponde a agregados absorbentes y débiles.

También se define como el cociente entre el peso de las partículas dividido entre el volumen de los sólidos únicamente, es decir no incluye los vacíos entre ellas. Su valor para agregados normales oscila entre 2500 y 2750 Kg/m^3.

PROCEDIMIENTO (Método del Balón)

a) Cuartear el material y tomar de ella aproximadamente 3 Kg.
b) Luego remojar el material durante 24 hrs.
c) Al día siguiente, eliminar el agua del recipiente, para luego esparcir el material sobre una superficie plana.
d) Comprobar si la muestra esta saturado superficialmente seco (S.S.S.), para ello echar en un molde tronco-cónico metálico apisonando con 25 golpes sin compactar el material; si al levantar el molde la muestra queda exacta al modelo del molde, esto significa que falta secar, pero si queda desmoronado, significa que ha secado demasiado, y si quedara desmoronado parcialmente y de punta significa que la muestra esta S.S.S.

e) Pesar 500 gr. del material S.S.S. y luego echar en un balón de vidrio para determinar su volumen por desplazamiento.
f) Pesar el balón con el material S.S.S.
g) Echar agua hasta 500cm^3 a temperatura ambiente de 23°C; para luego sacar el aire que se encuentra en dicho frasco, para ello agitar suavemente el balón.
h) Pesar el balón con agua y material S.S.S.
i) Secar el material, colocar en el horno a una temperatura de 100°C a 110°C.
j) Determinar el peso de la muestra secada al horno.
k) Con los datos obtenidos determinar el Peso Especifico.

2.1.2.2 ABSORCIÓN NTP 400.021

Capacidad del agregado fino de absorber el agua en contacto con el. Al igual que el contenido de humedad esta propiedad influye en la cantidad de agua para la relación agua/cemento en el concreto. También se define como la diferencia en peso del material superficialmente seco y el peso del material secado al horno (24hr) todo dividido entre el peso seco y multiplicado por 100. El valor de absorción puede ser determinado a partir de los datos para el cálculo del peso especifico.

Los datos y cálculos del peso especifico y porcentaje de absorción se muestra en el cuadro N° 2.1

CUADRO N° 2.1

PESO ESPECIFICO Y PORCENTAJE DE ABSORCION

	Descripcion	Und	M1	M2	M3	Promedio
A	Psss	gr	500.00	500.00	500.00	500.00
B	Pprob+Pagua	gr	702.00	702.00	702.00	702.00
C	Volumen inicial	cc	507.00	506.00	505.00	506.00
D	Psss+Pprob+Pagua	gr	1201.50	1201.00	1200.50	1201.00
E	Volumen final	cc	696.00	695.00	694.00	695.00
F	Pagua+Pprob	gr	887.50	886.00	886.50	886.67
G	Pseco horno	gr	488.00	490.00	486.00	488.00
H	Volumen muestra (F-B)	cc	185.50	184.00	184.50	184.67
I	Pesp.sss (A/H)	gr/cc	2.70	2.72	2.71	2.71
J	Pesp. Aparente (G/H-(A-G))	gr/cc	2.81	2.82	2.85	2.83
K	Pesp.masa (G/H)	gr/cc	2.63	2.66	2.63	2.64
L	Absorcion (A-G/G*100)	%	2.46	2.04	2.88	2.46

2.1.2.3 PESO UNITARIO SUELTO Y COMPACTADO NTP 400.017

Resulta del cociente de dividir el peso de las partículas entre el volumen total incluyendo los vacíos. Este valor es un parámetro hasta cierto punto relativo, puesto que esta influenciado por los espacios vacíos entre partículas como consecuencia del acomodo entre estas. El peso unitario compactado se emplea generalmente para estimar las proporciones entre agregados; mientras que el peso unitario suelto para realizar conversiones de dosificaciones de mezcla en volumen. Este valor de peso unitario oscila entre 1500 y 1700 Kg/m^3.

PROCEDIMIENTO

a) Cuartear adecuadamente el material.
b) Llenar el balde metálico de (1/10) pie^3 con el material y enrasarlo; luego pesarlo para el calculo del peso unitario suelto.
c) Llenar el balde con material en 3 capas, en cada capa aplicar 25 golpes con una varilla de D=5/8" con punta lisa y redondeada; y con 60cm de longitud.
   Cuando se compacta la primera capa no golpear el fondo del recipiente con fuerza. Al compactar las ultimas capas solo se emplea la fuerza necesaria para que la barra penetre la ultima capa de agregado.
   Luego enrasar y pesar para obtener el peso unitario compactado.
d) Pesar el balde metálico.
e) A fin de determinar la capacidad o volumen exacto del balde se llenara con agua y se pesara; obteniendo el peso de agua contenida se determina el volumen.

Cálculos:

$$\underset{\text{P.U.S}}{\text{Peso Unitario Suelto}} = \frac{\text{Peso Suelto}}{\text{Volumen del Balde}}$$

$$\underset{\text{P.U.C}}{\text{Peso Unitario Compactado}} = \frac{\text{Peso Compactado}}{\text{Volumen del Balde}}$$

Los datos y cálculos del peso unitario suelto y peso unitario compactado se muestra en el cuadro N° 2.2

CUADRO N° 2.2

PESO UNITARIO

PESO UNITARIO SUELTO

	Descripcion	Und	M1	M2	M3	Promedio
A	Pbalde (1/10 p^3)	gr	2779.50	2779.50	2779.50	2779.50
B	Pbalde+Pmuestra	gr	7235.00	7277.00	7302.00	7271.33
C	Pmuestra (B-A)	gr	4455.50	4497.50	4522.50	4491.83
D	Volumen balde	cc	2831.68	2831.68	2831.68	2831.68
E	Peso unitario suelto (C/D)	gr/cc	1.573	1.588	1.597	1.586

P.U.S = 1.586

PESO UNITARIO COMPACTADO

	Descripcion	Und	M1	M2	M3	Promedio
A	Pbalde (1/10 p^3)	gr	2779.50	2779.50	2779.50	2779.50
B	Pbalde+Pmuestra	gr	7989.00	7973.50	7960.00	7974.17
C	Pmuestra (B-A)	gr	5209.50	5194.00	5180.50	5194.67
D	Volumen balde	cc	2831.68	2831.68	2831.68	2831.68
E	Peso unitario compact (C/D)	gr/cc	1.840	1.834	1.829	1.834

P.U.C = 1.834

2.1.2.4 GRANULOMETRÍA NTP 400.012

Es la representación numérica de la distribución volumétrica de las partículas por tamaños.

Como seria sumamente difícil de medir el volumen de los diferentes tamaños de partículas, se usa una manera indirecta, la cual es tamizarlas por una serie de mallas de aberturas conocidas y pesar los materiales retenidos refiriéndoles en porcentajes con respecto al peso total. Los valores hallados se representan gráficamente en un sistema coordenado semi-logarítmico que permite apreciar la distribución acumulada.

El Reglamento Nacional de Construcción especifica la Granulometría de la arena en concordancia con la norma ASTM C-33.

En el cuadro N° 2.3 se puede observar los requisitos granulométricos para el agregado fino.

CUADRO N°2.3

HUSO UTILIZADO NORMA ASTM C-33

Malla	%Que Pasa
3/8"	100
N°4	95 - 100
N°8	80 - 100
N°16	50 - 85
N°30	25 - 60
N°50	10 - 30
N°100	10 - 30

Se tiene cuidado con las arenas finas, puesto que estos originan concretos de mayor costo dad la exigencia de agua, esto puede corregirse disminuyendo la relación de arena/piedra.

PROCEDIMIENTO:

a) Pesar 500 gr. libre de impurezas.
b) Antes de proceder al tamizado de la muestra, verificar el estado y orden de las mallas. Esta deberá tener el orden de acuerdo al tamaño de la abertura de la siguiente manera: N° 4, N° 8, N° 16, N° 30, N° 50, N° 100 y fondo.
c) Zarandear por espacio de 2 minutos.
d) Luego pesar el material retenido en cada malla.
e) Con los datos obtenidos realizar los cálculos correspondientes de acuerdo al cuadro N° 2.4 que se presenta. Se muestra asimismo el gráfico N° 2.1 correspondiente de acuerdo a los husos para este agregado.

CUADRO N° 2.4

GRANULOMETRIA

Muestra: 500 gr

Tamiz	Peso Retenido en Cada Malla (gr)			Promedio	%Retenido	%Retenido Acumulado
	M1	M2	M3			
N° 4	4.00	1.50	3.00	2.83	0.57	0.57
N° 8	110.50	89.50	108.50	102.83	20.57	21.13
N° 16	126.50	117.50	118.00	120.67	24.13	45.27
N° 30	91.00	94.00	88.50	91.17	18.23	63.50
N° 50	79.00	90.50	83.00	84.17	16.83	80.33
N° 100	44.00	59.00	49.00	50.67	10.13	90.47
FONDO	45.00	48.00	50.00	47.67	9.53	100.00
TOTAL	500.00	500.00	500.00	500.00		

Módulo de Finura : Suma%RA(N°4,N°8,N°16,N°30,N°50,N°100) / 100

M.F. = 3.01

2.1.2.5 CONTENIDO DE HUMEDAD ASTM C - 566

Es la cantidad de agua que contiene el agregado fino, esta característica al igual que la absorción son importantes ya que influyen en la determinación final de la cantidad de agua en el concreto.

PROCEDIMIENTO:

a) Pesar 500gr del material libre de impurezas en estado natural.
b) Colocar en un recipiente metálico y secar al horno durante 24 horas a 100°C a 110°C.
c) Pesar la muestra secada al horno y determinar la cantidad de agua en la muestra por diferencia de peso, para expresarla en porcentaje del peso seco.

Realizar los respectivos cálculos de acuerdo al cuadro N° 2.5 adjunto.

CUADRO N°2.5

CONTENIDO DE HUMEDAD

Muestra: 500 gr

	Descripción	Und	M1	M2	M3	Promedio
A	Pmuestra humeda	gr	500.00	500.00	500.00	500.00
B	Pmuestra seca horno	gr	490.00	494.00	494.00	492.67
C	Cont. Humedad (A-B/B)	%	2.04	1.21	1.21	1.49

Promedio: ***C.H = 1.49***

2.1.2.6 CANTIDAD QUE PASA LA MALLA N° 200 NTP 400.018

Es el porcentaje de material muy fino, tal como arcilla, limo, etc. que existe en el agregado, los valores altos disminuyen la resistencia del concreto, debido a que afectan a la adherencia entre los agregados y la pasta y consumen mayor cantidad de agua.

Los datos y cálculos se muestran el cuadro N° 2.6

LIMITES:

Parámetros definidos: Max. 5%

Casos especiales : Max. 3%

CUADRO N°2.6

MATERIAL QUE PASA LA MALLA N°200

	Descripcion	Und	M1	M2	M3	Promedio
A	Pmuestra seca horno	gr	500.00	500.00	500.00	500.00
B	Pmuestra seca lavada	gr	476.00	478.00	475.00	476.33
C	Porc. de finos (A-B/A)	%	4.80	4.40	5.00	4.73

Promedio: ***% Finos 4.73***

2.2 AGREGADO GRUESO

2.2.1 DEFINICIÓN

Es el retenido en el tamiz 4.75 mm (N° 4) proveniente de la desintegración natural o mecánica de la roca, que cumple con los limites establecidos para su empleo.

El agregado grueso debe ser piedra triturada gravas naturales limpias, libres de polvo superficial y debe cumplir con los requisitos especificados en la NORMA ASTM C-33, excepto en cuanto a la granulometría.

Por economía el contenido de huecos del agregado debe mantenerse lo más bajo posible, entre 38 y 48%. El agregado grueso debe ser bien graduado usándose el mayor tamaño que pueda transportarse y colocarse económicamente en las cimbras, sin segregación excesiva.

Tomando en consideración la disponibilidad de los agregados por tamaño, tipo de construcción de que se trata.

CANTERA.- El agregado utilizado es el proveniente de la Cantera "GLORIA"

2.2.2 CARACTERÍSTICAS FÍSICAS

2.2.2.1 PESO ESPECÍFICO NTP 400.022

Es la relación a una temperatura estable, de la masa de un volumen unitario del material, a la masa del mismo volumen de agua destilada, libre de gas.

Además el peso especifico es un indicador de la calidad, ya que los valores elevados corresponden a materiales de buen comportamiento, mientras que el peso especifico de valor bajo generalmente corresponde a agregados absorbentes y débiles.

También se define como el cociente entre el peso de las partículas dividido entre el volumen de los sólidos únicamente, es decir no incluye los vacíos entre ellas.

Su valor para agregados normales oscila entre 2500 y 2750 Kg/m^3.

PROCEDIMIENTO

a) Seleccionar el material por el método del cuarteo, aproximadamente 6 Kg del agregado, rechazando todo material que pasa por el tamiz ITINTEC 4.75 mm (N°4).
b) Después de eliminado todas las impurezas polvo u otros materiales extraños de la superficie del material se seca el material y se pone a remojar en una vasija con agua por espacio de 24 horas.
c) Al día siguiente escurrir el agua del recipiente y echar el material sobre una mesa, enseguida con ayuda de una franela secar la superficie del a agregado, a fin de tener material S.S.S.
d) Pesamos 5 Kg del material secado.
e) Calibrar la Balanza hidrostática sin agua, llenar de agua el recipiente, y pesar la canastilla dentro del agua, anotar el peso cuando deje de gotear la salida del deposito de la balanza hidrostática.
f) Pesar el material dentro de la canastilla sumergida.
g) Poner el material pesado en un recipiente metálico a fin de colocar este dentro del horno durante 24horas.
h) Obtener el peso de la muestra secada al horno.
i) Realizar los cálculos de acuerdo al cuadro N° 2.7 adjunto.

2.2.2.2 ABSORCIÓN NTP 400.022

Es la capacidad de los agregados de llenar con agua los vacíos internos en las partículas. El fenómeno se produce por capilaridad, permeabilidad, etc.

Su valor se determina como el contenido de humedad en el estado saturado superficialmente seco del material y se expresa como porcentaje del peso seco.

Su valor puede ser determinado a partir de los datos del ensayo de peso especifico, como se muestra en el cuadro N° 2.7 siguiente:

CUADRO N°2.7

PESO ESPECIFICO Y PORCENTAJE DE ABSORCION

	Descripcion	Und	M1	M2	M3	Promedio
A	Psss	gr	500.00	500.00	500.00	500.00
B	Pprob+Pagua	gr	702.00	702.00	702.00	702.00
C	Volumen inicial	cc	506.00	505.00	505.00	505.33
D	Psss+Pprob+Pagua	gr	1202.00	1201.50	1202.00	1201.83
E	Volumen final	cc	690.00	690.00	692.00	690.67
F	Pagua+Pprob	gr	882.00	880.50	886.50	883.00
G	Pseco horno	gr	498.50	497.50	497.00	497.67
H	Volumen muestra (F-B)	cc	180.00	178.50	184.50	181.00
I	Pesp.sss (A/H)	gr/cc	2.78	2.80	2.71	2.76
J	Pesp. Aparente (G/H-(A-G))	gr/cc	2.79	2.83	2.74	2.79
K	Pesp.masa (G/H)	gr/cc	2.77	2.79	2.69	2.75
L	Absorcion (A-G/G*100)	%	0.30	0.50	0.60	0.47

Promedio: ***Pesp.masa=*** ***2.75***
Absorcion = ***0.47***

2.2.2.3 PESO UNITARIO SUELTO Y COMPACTADO NTP 400.017

Resulta del cociente de dividir el peso de las partículas entre el volumen total incluyendo los vacíos. Este valor es un parámetro hasta cierto punto relativo, puesto que esta influenciado por los espacios vacíos entre partículas como consecuencia del acomodo entre estas. El peso unitario compactado se emplea generalmente para estimar las proporciones entre agregados; mientras que el peso unitario suelto para realizar conversiones de dosificaciones de mezcla en volumen. Este valor de peso unitario oscila entre 1,500 y 1,700 Kg/m^3.

PROCEDIMIENTO

a) Cuartear adecuadamente el material.
b) Llenar el balde metálico de (1/2) pie^3 con el material y enrasarlo; luego pesarlo para el calculo del peso unitario suelto.
c) Llenar el balde con material en 3 capas, en cada capa aplicar 25 golpes con una varilla de D=5/8" con punta lisa y redondeada; y con 60cm de longitud.

d) Se recomienda al compactar la primera capa no golpear el fondo del recipiente con fuerza. Al compactar las ultimas capas solo se emplea la fuerza necesaria para que la barra penetre la ultima capa de agregado.

e) Luego enrasar y pesar para obtener el peso unitario compactado. Pesar el balde metálico.

f) A fin de determinar la capacidad o volumen exacto del balde se llenara con agua y se pesara; obteniendo el peso de agua contenida se determina el volumen.

Cálculos:

$$\underset{\text{P.U.S}}{\text{Peso Unitario Suelto}} = \frac{\text{Peso Suelto}}{\text{Volumen del Balde}}$$

$$\underset{\text{P.U.C}}{\text{Peso Unitario Compactado}} = \frac{\text{Peso Compactado}}{\text{Volumen del Balde}}$$

Los datos y cálculos del peso unitario suelto y compactado del Agregado Grueso se presenta en el cuadro N° 2.8

CUADRO N°2.8

PESO UNITARIO

PESO UNITARIO SUELTO

	Descripcion	Und	M1	M2	M3	Promedio
A	Pbalde (1/2 p^3)	gr	11800.00	11800.00	11800.00	11800.00
B	Pbalde+Pmuestra	gr	31500.00	31750.00	31750.00	31666.67
C	Pmuestra (B-A)	gr	19700.00	19950.00	19950.00	19866.67
D	Volumen balde	cc	14158.42	14158.42	14158.42	14158.42
E	Peso unitario suelto (C/D)	gr/cc	1.391	1.409	1.409	1.403

Promedio: ***P.U.S = 1.403***

PESO UNITARIO COMPACTADO

	Descripcion	Und	M1	M2	M3	Promedio
A	Pbalde (1/2 p^3)	gr	11800.00	11800.00	11800.00	11800.00
B	Pbalde+Pmuestra	gr	33650.00	34000.00	33900.00	33850.00
C	Pmuestra (B-A)	gr	21850.00	22200.00	22100.00	22050.00
D	Volumen balde	cc	14158.42	14158.42	14158.42	14158.42
E	Peso unitario suelto (C/D)	gr/cc	1.543	1.568	1.561	1.557

Promedio: ***P.U.C = 1.557***

2.2.2.4 GRANULOMETRÍA NTP 400.012

Es la representación numérica de la distribución volumétrica de las partículas por tamaños.

Como seria sumamente difícil de medir el volumen de los diferentes tamaños de partículas, se usa una manera indirecta, la cual es tamizarlas por una serie de mallas de aberturas conocidas y pesar los materiales retenidos refiriéndoles en porcentajes con respecto al peso total. Los valores hallados se representan gráficamente en un sistema coordenado semi-logarítmico que permite apreciar la distribución acumulada.

Para el agregado grueso debe tenerse en cuenta lo siguiente:

Tamaño Máximo

Es la menor malla por la que pasa el 100% del material.

Tamaño Nominal Máximo

Existen dos definiciones:

$T^{n}max$

Malla que pasa del 95-100% o en el que se produce el primer retenido.

$T^{n}max$

Diámetro del tamiz inmediato superior al que retiene el 15% o más en forma acumulada del material.

En la presente tesis de investigación utilizaremos la ultima definición.

MÓDULO DE FINURA (M.F.) NTP 400.012

Se define como la suma de los porcentajes retenidos acumulativos de la serie Standard hasta el tamiz N° 100 y esta cantidad se divide entre 100; este valor que es proporcional al promedio logarítmico del tamaño de partículas de una cierta distribución granulométrica, sirve para caracterizar cada agregado independientemente o la mezcla de agregados en conjunto.

Módulo de Finura : $\dfrac{\text{Suma\%RA(1 1/2",3/4",3/8",N°4,N°8,N°16,N°30,N°50,N°100)}}{100}$

PROCEDIMIENTO

a) Pesar 10 kg. de piedra.
b) Limpiar las mallas y verificar el correcto orden de las mallas.
c) Zarandear el material por un tiempo de 2 minutos.
d) Pesar el material retenido en cada malla.
e) Realizar los cálculos de acuerdo al siguiente cuadro N° 2.9.

CUADRO N°2.9

GRANULOMETRIA

Muestra: 6000 gr

Tamiz	Peso Retenido en Cada Malla (gr)			Promedio	%Retenido	%Retenido Acumulado
	M1	M2	M3			
1"	0.00	0.00	0.00	0.00	0.00	0.00
3/4"	1158.00	1360.00	1493.00	1337.00	22.28	22.28
1/2"	2512.00	2639.00	2540.00	2563.67	42.73	65.01
3/8"	1677.00	1523.00	1495.00	1565.00	26.08	91.09
1/4"	628.00	449.00	454.00	510.33	8.51	99.60
FONDO	25.00	29.00	18.00	24.00	0.40	100.00
TOTAL	6000.00	6000.00	6000.00	6000.00		

Tamaño Máximo: 1"
Tamaño Máximo Nominal: 1"
Módulo de Finura :

Suma%RA(1 1/2",3/4",3/8",N°4,N°8,N°16,N°30,N°50,N°100) / 100
M.F. = 7.13

HUSO UTILIZADO NORMA ASTM C-33 N° 57

% Que Pasa	100	95 A 100	25 A 60	0 A 10	0 A 5
Malla	1 1/2"	1"	1/2"	N° 4	N° 8

2.2.2.5 CONTENIDO DE HUMEDAD ASTM C - 566

Es la cantidad de agua que contiene el agregado fino, esta característica al igual que la absorción es importante ya que influye en la determinación final de la cantidad de agua en el concreto.

PROCEDIMIENTO:

a) Pesar 1000gr del material libre de impurezas en estado natural.
b) Colocar en un recipiente metálico y secar al horno durante 24 horas a 100°C a 110°C.
c) Pesar la muestra secada al horno y determinar la cantidad de agua en la muestra por diferencia de peso, para expresarla en porcentaje del peso seco.

Realizar los respectivos cálculos de acuerdo al cuadro adjunto N° 2.10.

CUADRO N°2.10

CONTENIDO DE HUMEDAD

	Descripcion	Und	M1	M2	M3	Promedio
A	Pmuestra humeda	gr	500.00	500.00	500.00	500.00
B	Pmuestra seca horno	gr	499.00	498.80	498.20	498.67
C	Cont. Humedad (A-B/B)	%	0.20	0.24	0.36	0.27

Promedio: ***C.H.*** = ***0.27***

Se presenta a continuación en el cuadro N° 2.11 un resumen de las propiedades físicas de los Agregados Fino y Grueso, además del grafico N° 2.1, donde se muestra el análisis granulométrico con sus respectivos Husos.

CUADRO N°2.11

RESUMEN DE LAS CARACTERISTICAS FISICAS

Descripcion	Und	Agregado	
		Fino	Grueso
Peso Unitario Suelto	gr/cc	1.586	1.403
Peso Unitario Compactado	gr/cc	1.834	1.557
Peso Específico de Masa	gr/cc	2.643	2.750
Peso Específico de Masa Saturada Superficialmente Seco	gr/cc	2.708	2.763
Peso Específico Aparente	gr/cc	2.826	2.786
Porcentaje de Absorción	%	2.460	0.469
Contenido de Humedad	%	1.490	0.267
Superficie Específica	cm^2/gr	47.860	1.112
Material que Pasa la malla N° 200	%		
Tamaño máximo	Plg	—	1"
Tamaño máximo Nominal	Plg	—	1"
Módulo de Finura	M.F.	3.013	7.134

GRAFICA 2.1

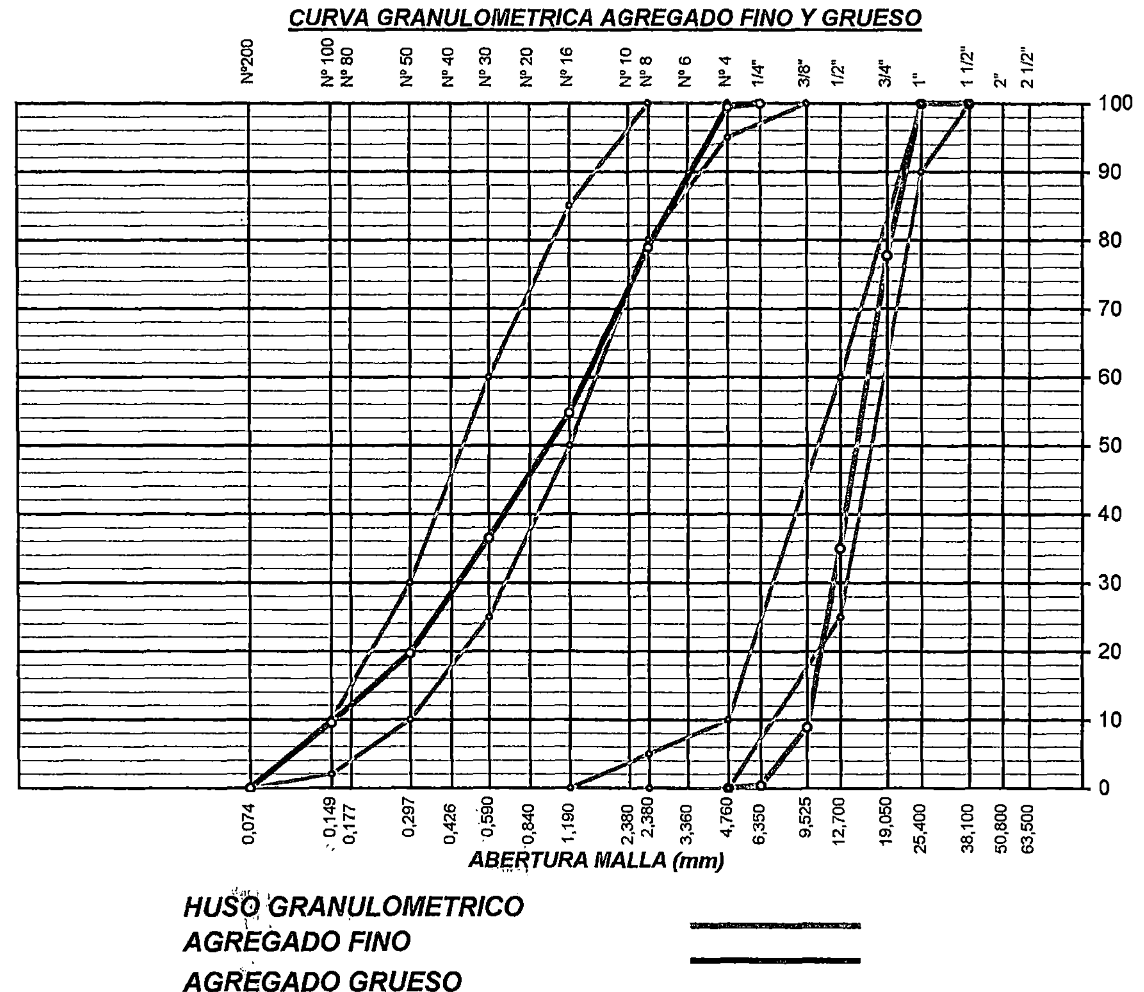
PROYECTO: TESIS

BACH: TARAZONA TINOCO JAIME CESAR

MATERIAL:
AGREGADO FINO :
AGREGADO GRUESO :

PROCEDENCIA:
CANTERA LA GLORIA
CANTERA LA GLORIA

MALLAS SERIE AMERICANA	ANÁLISIS GRANULOMÉTRICO						
	ABERT. (mm)	AGREG FINO		AGREG GRUESO		ESPECIFICACIONES	
						AGREG FINO	AGREG GRUESO
		% RET. ACUM.	% PASA ACUM.	% RET. ACUM.	% PASA ACUM.		
2 1/2"	63.500						
2"	50.800						
11/2"	38.100			0	100		100
1"	25.400			0.00	100		90 - 100
3/4"	19.050			22.28	77.72		
1/2"	12.700			65.01	34.99		25 - 60
3/8"	9.525			91.09	8.91	100	
1/4"	6.350		100	99.60	0.40		
N° 4	4.760	0.57	99.43	100	0.00	95 - 100	0 - 10
N° 6	3.360						
N° 8	2.380	21.13	78.87			80 - 100	0 - 5
N° 10	2.000						
N°16	1.190	45.27	54.73			50 - 85	0
N° 20	0.840						
N° 30	0.590	63.50	36.50			25 - 60	
N° 40	0.426						
N° 50	0.297	80.33	19.67			10 - 30	
N° 80	0.177						
N° 100	0.149	90.47	9.53			2 - 10	
N° 200	0.074	100	0.00				



Tesis: "Estudio del comportamiento del concreto de mediana a baja resistencia, con la incorporación de Fibras de Acero y CPT I Andino"

2.3 AGREGADO GLOBAL

Para la determinación de la optima relación porcentual entre el agregado fino y grueso se usó el método de la "compacidad" y de resistencia máxima.

Para la evaluación granulométrica nos remitiremos a los husos DIN 1045 para el agregado global.

Para concreto mas trabajable

- Concreto de mejor trabajabilidad del huso "A" al "B"
- Concretos de trabajabilidad aceptable del huso "B" al "C"

Los requisitos granulométricos para el Agregado Global se pueden observar en el Cuadro N° 2.12

CUADRO N°2.12

HUSO DIN 1045

TAMAÑO MAXIMO = 32mm			
MALLA (mm)	**FRACCION QUE PASA**		
	A	**B**	**C**
31.50	100	100	100
16.00	62	80	89
8.00	38	62	77
4.00	23	47	65
2.00	14	37	53
1.00	8	28	42
0.50			
0.25	2	8	15

2.3.1 MÉTODO DE COMPACIDAD

El objetivo de este método es de buscar el mejor acomodo que puedan tener los agregados entre sí.

Este método consiste en mezclar los agregados (piedra y arena) en diferentes porcentajes a fin de obtener para cada porcentaje un valor de Peso Unitario Compactado (PUC); con estos datos realizar un gráfico de PUC vs % Arena y determinar en ella el mayor PUC; la cual nos quiere decir que tiene mejor acomodo entre las partículas.

En la presente tesis se realizaron en total cinco combinaciones de porcentajes de arena y piedra, las cuales se observan en el Cuadro N° 2.13

CUADRO 2.13

PESO UNITARIO COMPACTADO DE LA COMBINACION

Wtotal (gru+fin) = 50 Kg
Wvasija = 11.8 Kg
Vvasija = 1/2 pie^3

Agreg %	
Fino	**Grueso**
40	60
45	55
50	50
55	45
60	40

Comb	Fino	Grueso	W(vasija+muestra)			Wprom	Wmuestra	P.U.C.
	(Kg)	(Kg)	M-1	M-2	M-3	(Kg)	(Kg)	(Kg/m^3)
1	20.00	30.00	38.80	39.80	39.60	39.40	27.60	1949.37
2	22.50	27.50	41.25	40.65	40.85	40.92	29.12	2056.49
3	25.00	25.00	41.20	41.30	41.10	41.20	29.40	2076.50
4	27.50	22.50	40.80	41.10	40.90	40.93	29.13	2057.67
5	30.00	20.00	40.40	39.95	40.35	40.23	28.43	2008.23

Para toda las combinaciones tomamos como base 50 kilos del total de la mezcla obteniendo pesos para el agregado fino y el agregado grueso de acuerdo a los porcentajes correspondientes, para cada combinación se realizaron tres muestras obteniendo un peso promedio de la muestra y posteriormente el peso unitario compactado

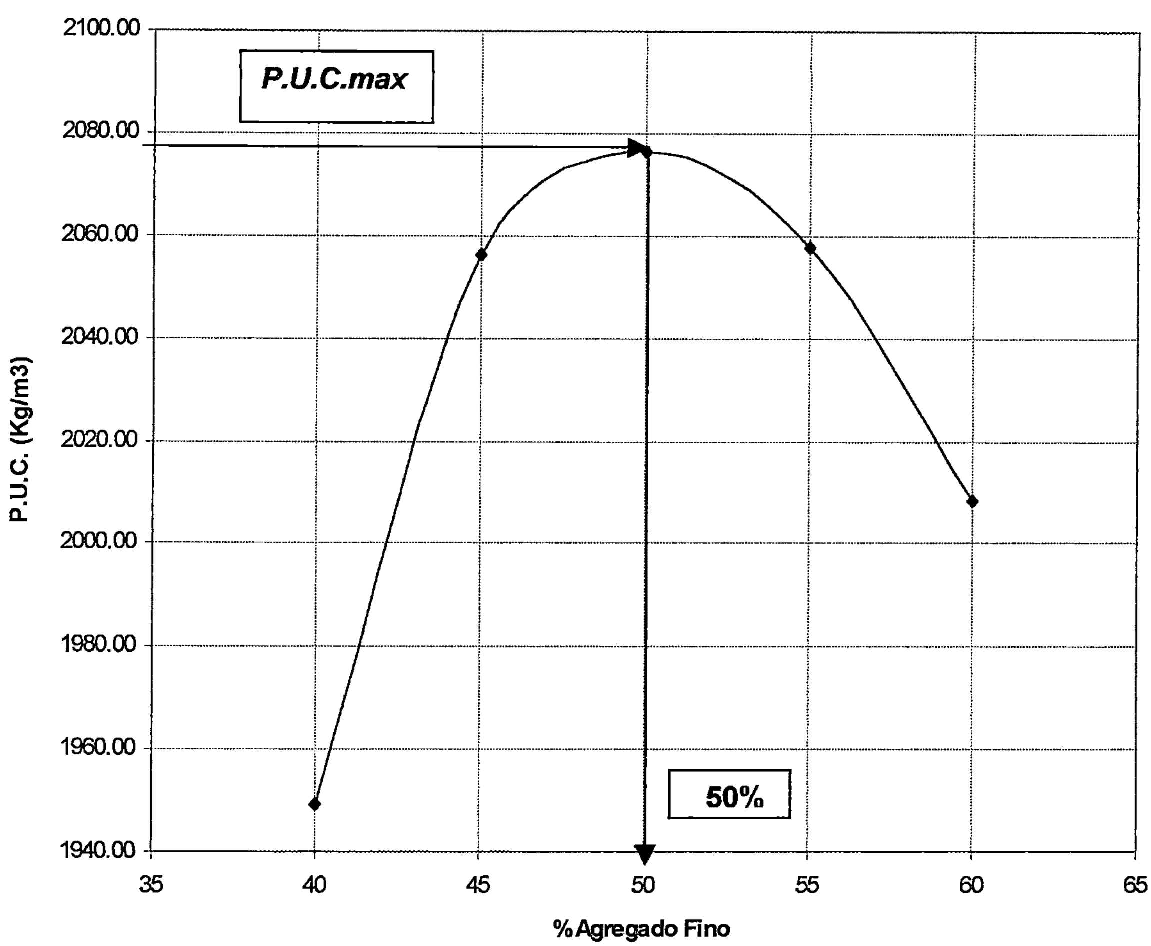
Para obtener el mayor P.U.C. se realiza una grafica de %Agregado Fino vs P.U.C. para cada combinación según el cuadro N° 2.14

La grafica N° 2.2 muestra el máximo P.U.C. y su respectivo %Agregado Fino

CUADRO 2.14

%A.F.	PUC
40	1949.37
45	2056.49
50	2076.50
55	2057.67
60	2008.23

GRAFICA 2.2
ENSAYO DE MAXIMA COMPACTACION DEL AREGADO GLOBAL



En el Gráfico N° 2.2, observamos que la relación de 50% de piedra y 50% de arena son los que tienen mayor peso unitario compactado; como sabemos este método de compacidad es una aproximación a la optima relación arena piedra.

2.3.2 MÉTODO DE LA RESISTENCIA MÁXIMA

Del grafico de puede observar que para 50% de arena y 50% de piedra en peso, se obtiene el mayor peso unitario compactado de la combinación de agregados. Este es el primer indicador de los porcentajes de agregados en la mezcla pero no es el definitivo, debiéndose verificar la máxima resistencia en estos intervalos de combinación de agregado global ± 3%.

A fin de lograr la proporción optima del Agregado Global, habrá que observarse las principales propiedades del concreto, que satisfagan las condiciones de trabajabilidad y resistencia. Para ello se realizará tres mezclas de prueba, considerando un asentamiento de 3-4", las cantidades de agua y relación (a/c) = 0.65 constante.

En total se prepararon tres probetas para cada relación arena/piedra, para finalmente sacar un promedio; se curaron por espacio de 7 días y se procedió a ensayar en la maquina de compresión obteniéndose los resultados mostrados en el Cuadro N° 2.15

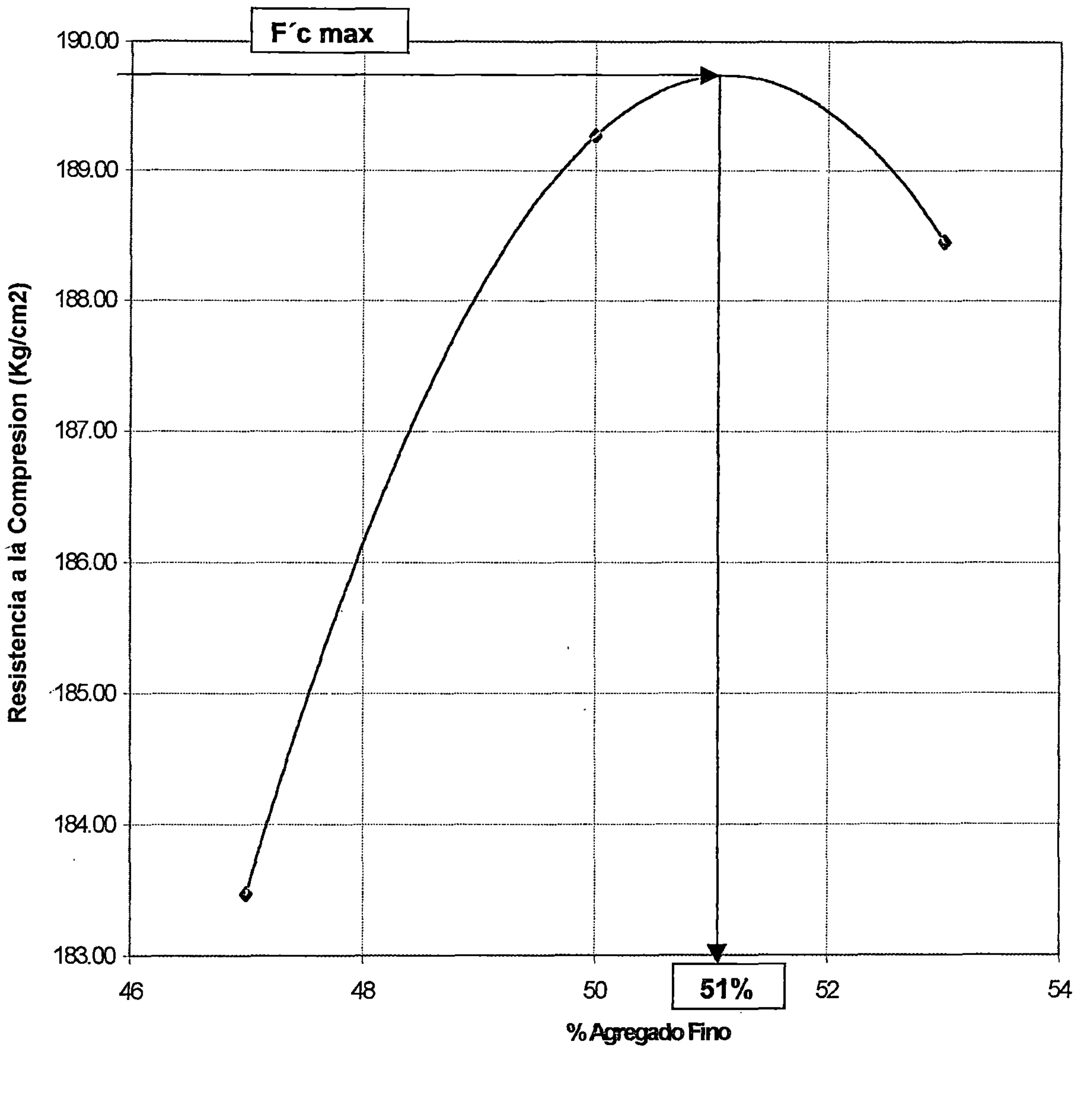
CUADRO N° 2.15

<u>RESISTENCIA DEL CONCRETO A LOS 7 DIAS</u>

% A.F.	MUESTRA	Diámetro	Area (cm2)	Carga	fc
47%	M - 1	14.90	174.00	30200	173.56
	M - 2	14.90	174.00	33400	191.95
	M - 3	14.80	172.00	31800	184.88
		f'c =	**183.47**		
50%	M - 1	14.90	174.00	32400	186.21
	M - 2	14.90	174.00	33400	191.95
	M - 3	14.90	174.00	33000	189.66
		f'c =	**189.27**		
53%	M - 1	14.90	174.00	31400	180.46
	M - 2	14.80	172.00	33800	196.51
	M - 3	14.80	172.00	32400	188.37
		f'c =	**188.45**		

Con los datos obtenidos se procede a realizar la curva de ensayo de resistencia a la compresión a los 07 días como muestra la grafica N° 2.3

GRAFICA N°2.3
CURVA DE ENSAYO DE RESISTENCIA A LA COMPRESIÓN



En el Gráfico N° 2.3 observamos que la relación de 51% de arena y 49% de piedra, tiene mejor resistencia que las otras tres.

Por lo tanto a partir de estos porcentajes y a partir de las granulometrías parciales de arena y piedra se elabora la granulometría del agregado global.

En el Cuadro N° 2.16 y Gráfico N° 2.4 se muestra los detalles de calculo y el análisis granulométrico del agregado global respectivamente.

CUADRO N°2.16

GRANULOMETRIA

Norma : Huso Din 1045
Cantera: Gloria
Muestra: 50 kg

Tamiz	% Retenido Acumulado		
	Fino	Grueso	51%Fi+49%Gr
1"		0.00	0.00
3/4"		22.28	10.92
1/2"		65.01	31.85
3/8"		91.09	44.63
1/4"		99.60	48.80
N° 4	0.57	100.00	49.29
N° 8	21.13		59.78
N° 16	45.27		72.09
N° 30	63.50		81.39
N° 50	80.33		89.97
N° 100	90.47		95.14
FONDO	100.00		100.00
TOTAL			

Módulo de Finura : Suma%AR(1 1/2",3/4",3/8",N°4,N°8,N°16,N°30,N°50,N°100) / 100

M.F. = 5.03

En consecuencia la relación porcentual optima agregado fino será 51% y del grueso 49% con la cual trabajaremos para los diseños de mezclas respectivos.

En este ítem solo se mostró los resultados de los ensayos a compresión del concreto, mas adelante se explicará con mayor detalle el diseño de mezclas (Capitulo N° 06).

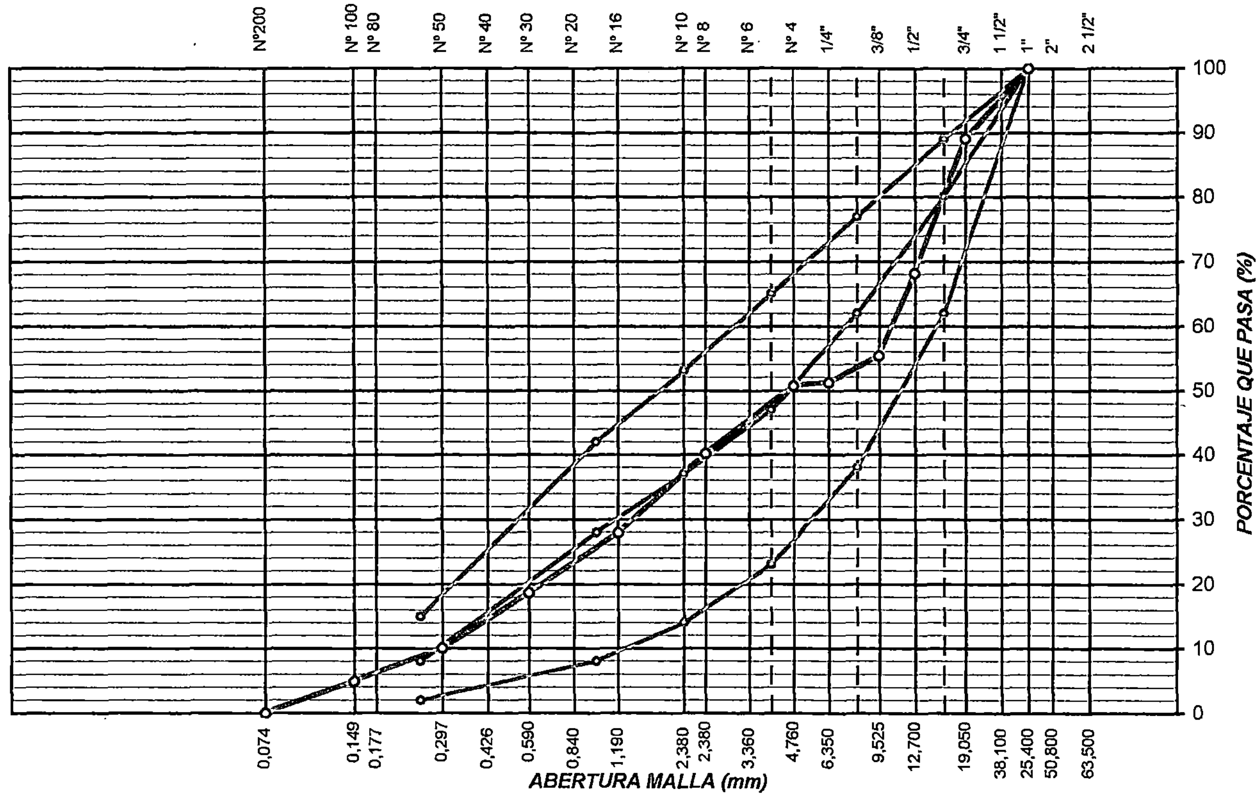
MALLAS SERIE AMERICANA	ANALISIS GRANULOMETRICO		
	ABERT (mm)	AGREGADO GLOBAL 51% A.F. + 49% A.G.	
		% RET	% PASA
2	50.800		
11/2"	38.100		
1"	25.400		
	31.500	0.00	100.00
3/4"	19.050	10.92	89.08
1/2"	12.700	31.85	68.15
3/8"	9.525	44.63	55.37
1/4"	6.350	48.80	51.20
N° 4	4.760	49.29	50.71
N° 6	3.360		
N° 8	2.380	59.78	40.22
N° 10	2.000		
N°16	1.190	72.09	27.91
N° 20	0.840		
N° 30	0.590	81.39	18.61
N° 40	0.426		
N° 50	0.297	89.97	10.03
N° 80	0.177		
N° 100	0.149	95.14	4.86
N° 200	0.074	100.00	0.00

Tamaño maximo **1"**

Modulo de Finura **5.03**

GRAFICA 2.4

CURVA GRANULOMETRICA DEL AGREGADO GLOBAL



Huso Din 1045

Agregado Global

Huso Din 1045			
Tamaño Maximo 32 mm			
Malla (mm)	Porcentaje Que Pasa		
	A	B	C
31.50	100	100	100
16.00	62	80	89
8.00	38	62	77
4.00	23	47	65
2.00	14	37	53
1.00	8	28	42
0.25	2	8	15

CAPÍTULO 3

AGUA

3.1 GENERALIDADES

Se sabe que el agua es el elemento indispensable para la hidratación del cemento y el desarrollo de sus propiedades, por lo tanto este elemento debe cumplir con ciertos requisitos para llevar a cabo su función en la combinación química.

El agua de mezcla tiene tres funciones principales:

1. Reaccionar con el cemento para hidratarlo.
2. Actuar como lubricante para contribuir a la trabajabilidad del conjunto y
3. Procurar la estructura de vacíos necesaria en la pasta para que los productos de hidratación tengan espacio para desarrollarse.

Por lo tanto la cantidad de agua que interviene en la mezcla de concreto es normalmente por razones de trabajabilidad, mayor de lo necesario para la hidratación del cemento.

El problema principal del agua reside en las impurezas y la cantidad de estas, que ocasionan reacciones químicas que alteran el comportamiento normal de la pasta de cemento.

Una regla empírica que sirve para estimar si determinada agua sirve para emplearse en la producción de concreto, consiste en establecer su habilidad para el consumo humano ya que si no daña al hombre no daña al concreto.

En este sentido es interesante distinguir el agua potable en términos de los requerimientos nominales establecidos por los organismos que regulan su producción y uso ya que los requerimientos aludidos son más exigentes de lo necesario.

3.2 PRINCIPALES REQUISITOS A CUMPLIR

La Norma NTP 339.088 establece como requisitos para agua de mezcla y curado:

CUADRO N°3.1

DESCRIPCION	LIMITE PERMISIBLE
Solidos en suspension	5000 ppm maximo
Matria organica	3 ppm maximo
Carbonatos y bicarbonatos alcalinos	1000 ppm maximo
Sulfatos	600 ppm maximo
Cloruros	1000 ppm maximo
PH	Entre 5.5 y 8.0

Existe evidencia experimental que el empleo de agua con contenidos individuales de cloruros, sulfatos y carbonatos sobre las 5000 ppm, ocasionan reducción de resistencias hasta del orden del 30% con relación a concretos con agua pura.

La materia orgánica por encima de las 1000 ppm reduce resistencia e incorpora aire.

El criterio que establece la Norma NTP 339.088 y el Comité ACI-318 para determinar la habilidad de determinada agua para emplearse en concreto, consiste en preparar cubos de mortero de acuerdo con la norma ASTM C-109, usando el agua dudosa y compararlos con cubos similares elaborados con agua potable. Si la resistencia en compresión a 7 y 28 días de los cubos con el agua en prueba no es menor del 90% de la de los cubos de control, se acepta el agua como apta para su uso en concreto.

Como dato interesante es una evidencia que en el Perú muy pocas "aguas potables" cumplen con las limitaciones nominales indicadas, sobre todo en lo que se refiere al contenido de sulfatos y carbonatos, sin embargo sirve para el consumo humano y consecuentemente para el concreto, por lo que no debe cometerse el error de establecer especificaciones para agua que luego no se puede satisfacer en la practica.

No existe un patrón definitivo en cuanto a las limitaciones en composición química que debe tener el agua de mezcla, ya que incluso aguas no aptas para el consumo humano sirven para preparar concreto y por otro lado depende mucho del tipo de cemento y las impurezas de los demás ingredientes.

Los efectos perniciosos que pueden esperarse del agua de mezcla con impurezas son: retardo en el endurecimiento, reducción de la resistencia, manchas en el concreto endurecido, eflorescencias, contribución a la corrosión del acero, cambios volumétricos, etc.

3.3 AGUA A UTILIZAR

El agua empleada en la presente Tesis para todas las mezclas realizadas es el agua potable, por lo tanto cumple con los requisitos establecidos en la Norma INTINTEC 339.088.

3.4 AGUA PARA CURADO

En general, el agua que es adecuada para mezcla también lo es para curado. Sin embargo, el hierro y la materia orgánica pueden ocasionar manchas, especialmente si el agua fluye lentamente sobre el concreto y se evapora con rapidez. En algunos casos la decoloración es insignificante y cualquier agua adecuada para mezcla, incluso de calidad ligeramente menor, es adecuado para curado. Sin embargo es esencial que esté libre de sustancias que ataquen al concreto endurecido, por ejemplo el CO_2 libre. El fluir de agua pura, proveniente de deshielo o de condensación, con poco CO_2 , disuelve el $Ca(OH)_2$ y provoca erosión de la superficie.

3.5 ATAQUE POR AGUA DE MAR

El agua marina contiene sulfatos y puede esperarse que ataque al concreto, esto por lo general no causa la expansión del concreto. La explicación reside en el hecho de que el yeso y la estringita son mas solubles en la solución del cloruro que en el agua, lo que significa que pueden extraerse con mayor facilidad por el agua marina. En consecuencia, no hay fractura sino sólo un pequeño incremento en la porosidad y, por tanto, una disminución en la resistencia.

También la expansión puede producirse como resultado de una presión ejercida por la cristalización de sales en los poros del concreto.

El concreto entre marcas de marea y sometido a secado y humedad alternantes es atacado severamente, mientras que el concreto que permanece inmerso es menos atacado. Sin embargo, el ataque por agua de mar es disminuido por el bloqueo de los poros en el concreto ocasionados por el deposito de hidróxido de magnesio.

En algunos casos, la acción del agua de mar en el concreto se acompaña por la acción destructiva del congelamiento, el impacto de las olas y la abrasión. Un daño adicional puede ser el causado por la ruptura del concreto que rodea el acero de refuerzo que se ha corroído debido a la acción electroquímica establecida por la absorción de sales que hace el concreto.

CAPÍTULO 4

FIBRAS DE REFUERZO

4.1 GENERALIDADES

El concreto de fibra reforzada es concreto hecho con cemento hidráulico, que contiene agregado fino, grueso, y fibras discontinuas. Las fibras pueden ser de material natural (asbestos, henequén, celulosa) o un producto manufacturado como vidrio, acero, carbón y polímero (polipropileno).

La pasta del cemento hidratado y del concreto es un tanto frágil. El propósito de reforzar la matriz de cemento base con fibras, es incrementar la resistencia a la tensión al retardar el crecimiento de grietas y aumentar la energía total absorbida antes de la separación total del espécimen al trasmitir esfuerzo a través de la sección agrietada, por lo que es posible una deformación mucho mayor mas allá del esfuerzo pico sin reforzamiento con fibra.

La cantidad de fibra usada es de 1 a 5% por volumen, y para darles efectividad como refuerzo de la resistencia a la tensión, elongación a la falla, y módulo de elasticidad, las fibras necesitan tener sustancialmente más altas las propiedades correspondientes de la matriz.

Un parámetro usado para caracterizar una fibra es la proporción adimensional, definido como diámetro equivalente que es la relación entre la longitud de la fibra dividido por su diámetro. El rango típico varia de 30 a 150 para longitud de dimensión de 6.35 mm a 76.2 mm.

Los diámetros de los tipos de fibras son:

- Acero : 0.25 a 0.80 mm
- Vidrio : 0.005 a 0.015 mm
- Polipropileno : 0.002 a 0.40 mm.

4.2 CLASIFICACIÓN DE LAS FIBRAS

El uso de la Fibras no es algo desconocido así tenemos la preparación del adobe con adiciones de paja. Las fibras empleadas en el concreto son elementos discontinuas

Las fibras actualmente empleadas pueden ser minerales, como las de asbesto y **vidrio**; orgánicas como las de algodón, poliéster, **polipropileno** y nilon; y metálicas, concretamente las de **acero**. Se han llevado a cabo experimentos satisfactorios con fibra de vidrio, filamentos cortos de polipropileno y fibras de acero usadas como refuerzo integral, dispersados aleatoriamente, investigando una cantidad de usos para evaluar la utilidad de los diferentes tipos de fibras y las diferentes características que pueden impartir al material en conjunto.

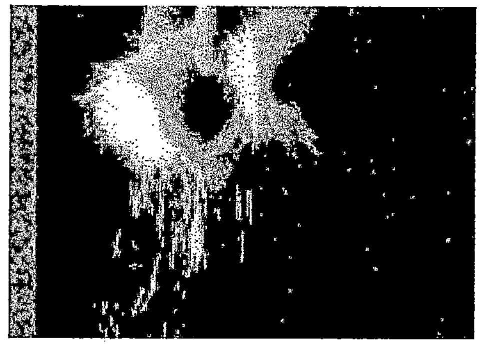
4.2.1 FIBRA DE VIDRIO

Las fibras de vidrio están sustituyendo, en sus aplicaciones, al asbesto, pero solo hasta hace poco se ha contado con esta fibra; a pesar de su inclusión satisfactoria en concretos y morteros, usando las técnicas tradicionales de moldeo se han experimentado dificultades al incluir suficiente fibra de vidrio en el concreto lanzado para mejorar algunas propiedades físicas. Sin embargo, puede decirse que se continua experimentando y que se han encontrado logros alentadores.

El concreto reforzado con fibra de vidrio se emplea para paneles decorativos prefabricados (planos o con forma) y para fachadas con propósitos arquitectónicos o de revestimientos.

En la figura 4.1 se muestra la fibra de vidrio

FIG. 4.1 FIBRA DE VIDRIO



4.2.2 FIBRA DE POLIPROPILENO

Son unos filamentos de gran resistencia estos filamentos contienen mono filamentos de polipropileno virgen al 100% .

El concreto lanzado usando fibra de polipropileno fue primero usado en Europa en 1968. Fue hasta finales de los 80's que se comenzaron a utilizar. Se usan con longitudes de 2 a 4 cm.

Las fibras de polipropileno pueden introducirse en mayor proporción en la mezcla en función del volumen. Cuando es mezclado dentro del concreto los atados de fibra se abren y se separan en filamentos individuales en diferentes direcciones.

4.2.3 FIBRA DE ACERO

Las Fibras metálicas, concretamente las de Acero son las que mas se emplean en el refuerzo del concreto.

La fibra de acero es fabricada por lo menos en tres procesos

a) Corte en frió del alambre,
b) División en hojas de acero
c) Extracción de una depósito de acero derretido

El mas común consiste en fabricarlas por corte .

El diámetro de los alambres está comprendido entre 0.25 y 0.80 mm.

La longitud de la fibras puede ser muy variable oscilando entre 10 y 75 mm.

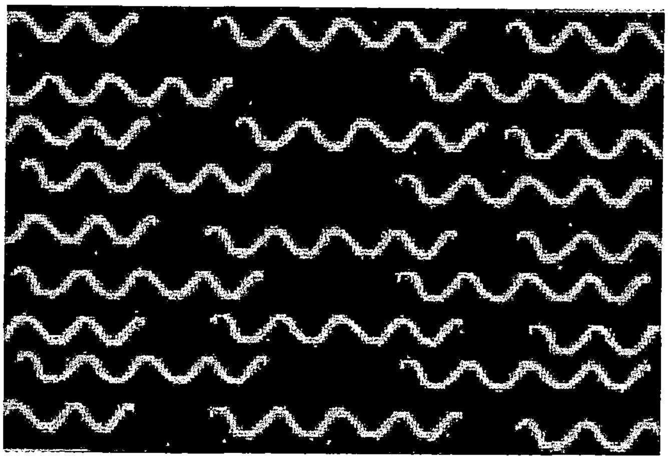
A efectos de comparación de unas fibras, con respecto a otras se ha establecido un parámetro numérico denominado diámetro equivalente, definido como la longitud de la fibra dividido por su diámetro. Los aspectos normales oscilan entre 30 y 150.

Para que cada fibra sea efectiva esta debe estar completamente embebida dentro de la mezcla; esto obliga a que la proporción de elementos finos a gruesos tenga que ser la adecuada.

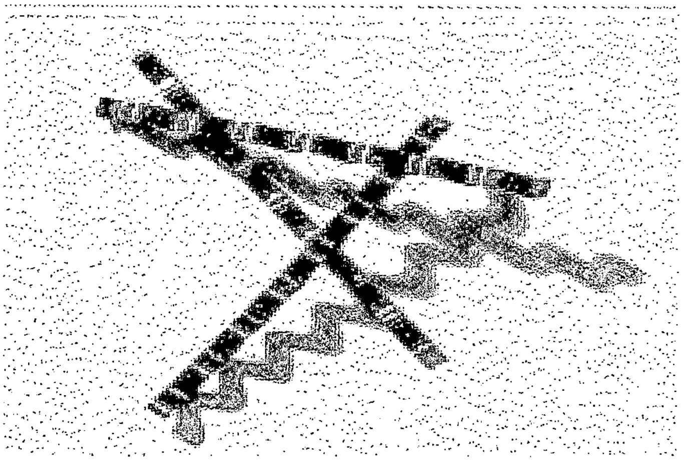
Cualquiera sea el método utilizado para el mezcla se debe obtener una dispersión uniforme de las fibras y eliminar los peligros de segregación y de formación de bolas o "erizos" de fibras, que esta relacionado con muchos parámetros, principalmente con el aspecto, el porcentaje de fibras, tamaño máximo del agregado grueso, granulometría, relación agua/cemento y sistema de mezclado.

A continuación se muestra en la figura 4.2, los diferentes tipos de Fibras de Acero,

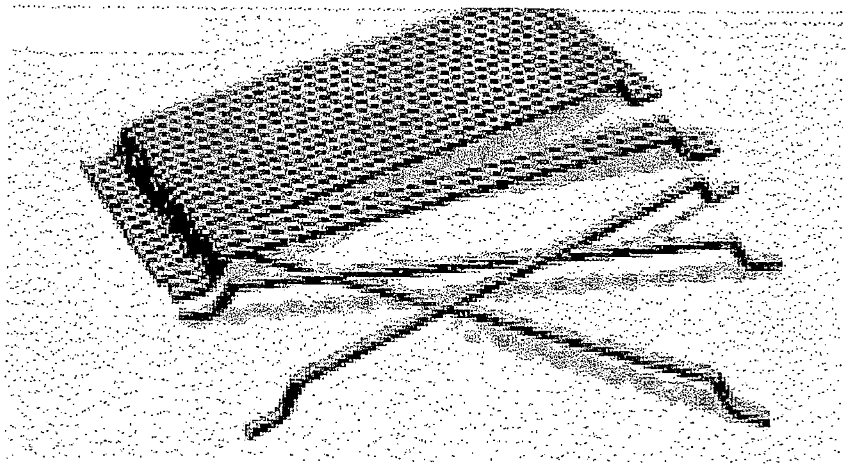
FIG. 4.2 TIPOS DE FIBRAS DE ACERO



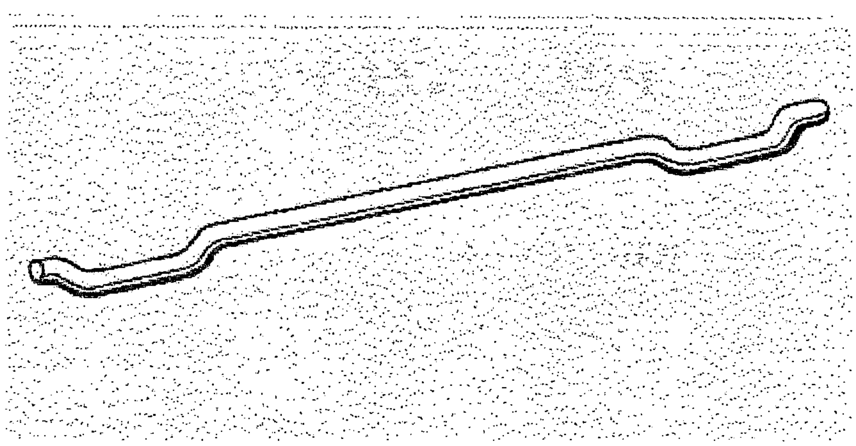
FIBRAS ONDULADAS Fig.(a)



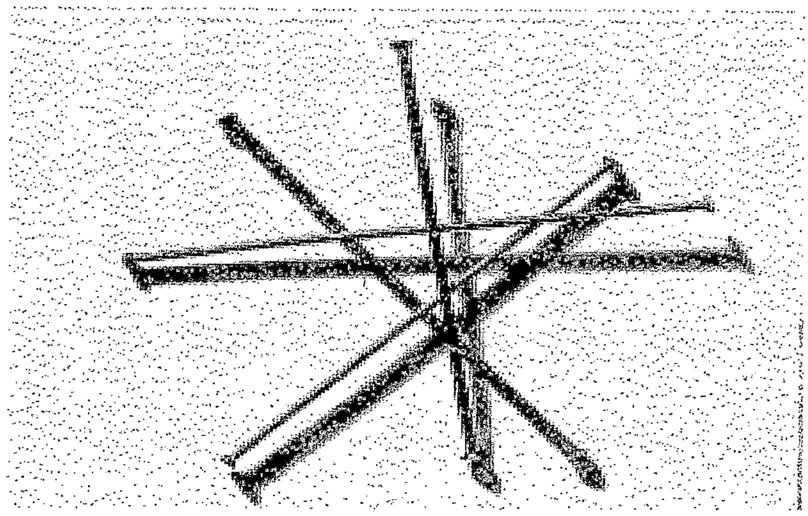
FIBRAS ONDULADAS Fig.(b)



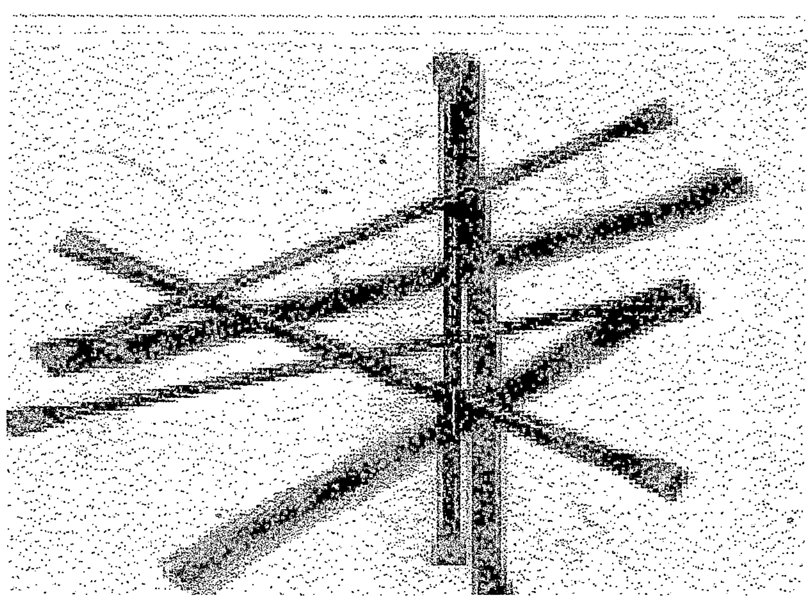
FIBRAS LISAS Fig.(c)



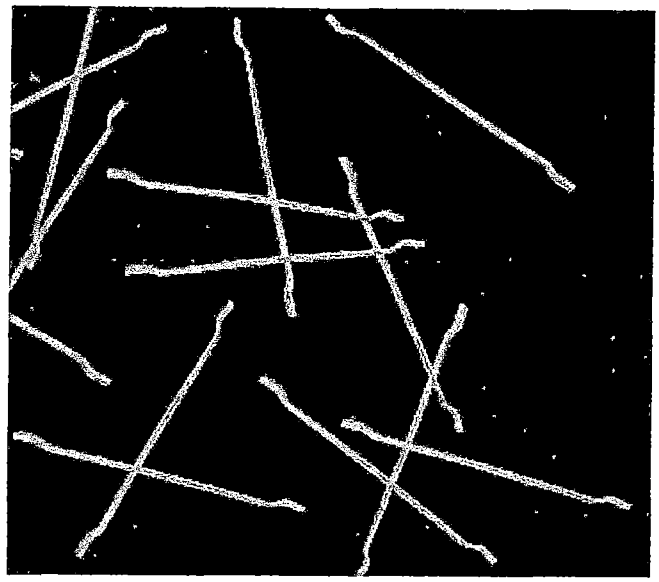
FIBRAS LISAS Fig.(d)



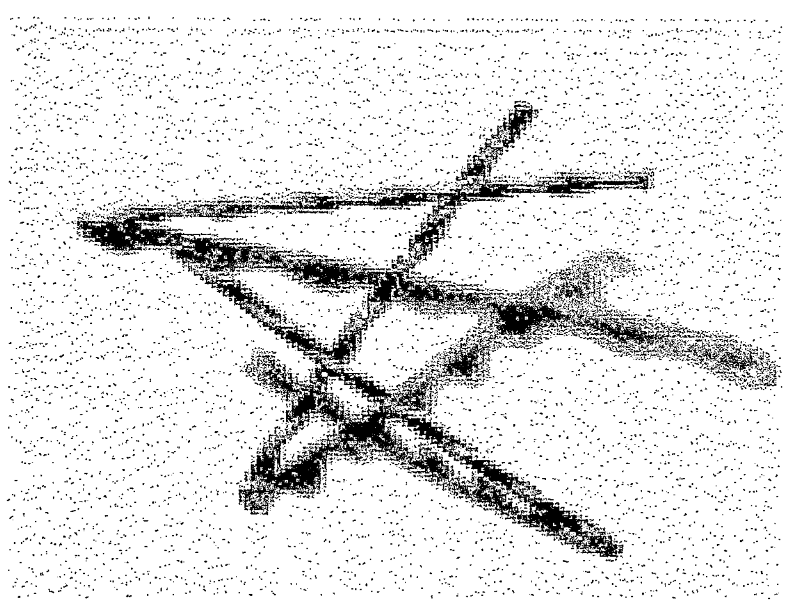
FIBRAS LISAS Fig.(e)



FIBRAS LISAS Fig.(f)



FIBRAS LISAS Fig.(g)



FIBRAS CORRUGADAS Fig.(h)

4.3 FIBRA DE ACERO INSONEX

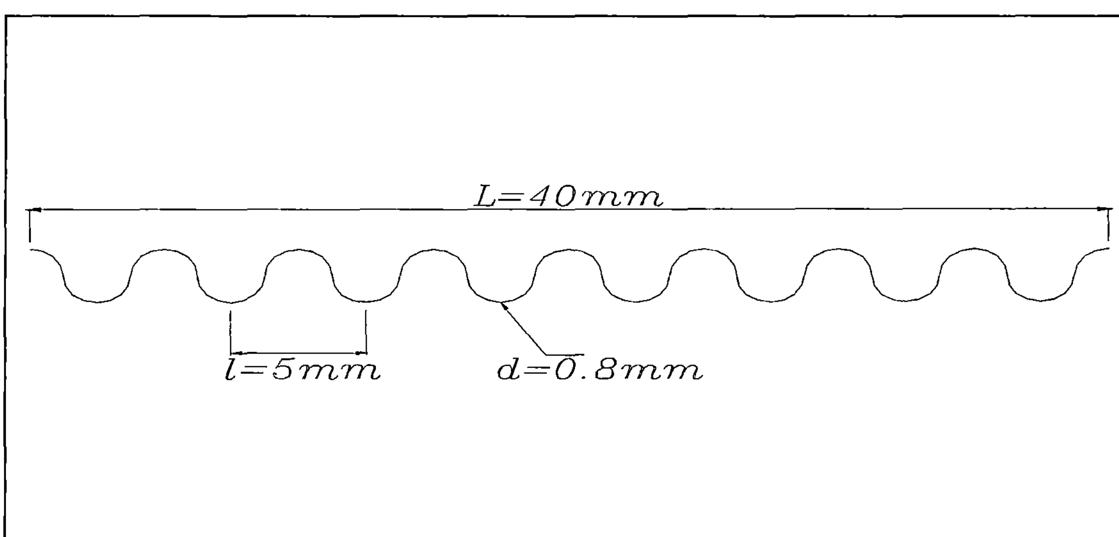
4.3.1 DESCRIPCIÓN

Las fibras de acero Insonex, son pequeños segmentos ondulados de longitud 40 mm, diámetro 0.80 mm, diámetro equivalente igual a 50 y otras características de dimensión que se muestra en el siguiente cuadro. En la figura 4.4 se muestra la geometría de la fibra.

DIMENSIONES	
Forma	Ondulada
Diámetro	D = 0.80 mm
Longitud	L = 40 mm
Altura de la onda	w = 0.65 mm
Longitud de la onda	l =5 mm
Resistencia mínima a la tracción del alambre	R = 76.5 kg/mm^2

FORMA DE SUMINISTRO	
Cajas	40 kg

FIG. 4.4 FIBRA DE ACERO INSONEX



En la figura 4.5 se muestra el mezclado de la fibra, en la figura 4.6 concreto con fibra de acero Insonex y en la figura 4.7, figura 4.8 y figura 4.9 algunas aplicaciones.

FIG. 4.5 MEZCLADO DE LA FIBRA

FIG. 4.6 CONCRETO CON FIBRA DE ACERO

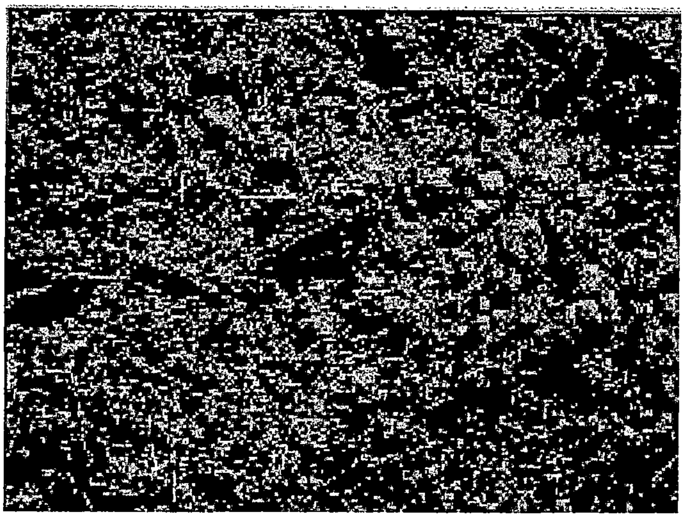


FIG. 4.7 CONCRETO CON FIBRA DE ACERO EN VIGAS

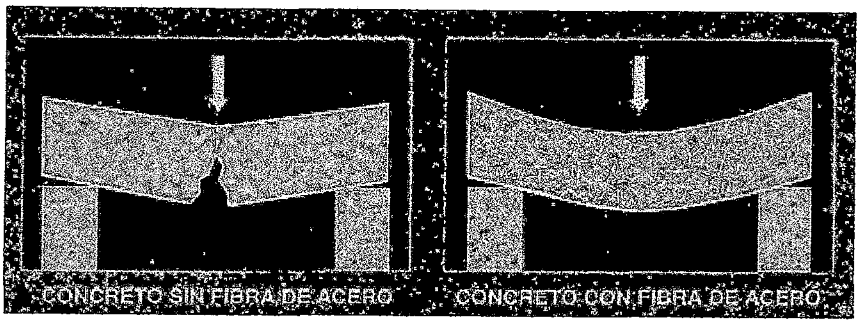


FIG. 4.8 FIBRA DE ACERO EN PAVIMENTOS

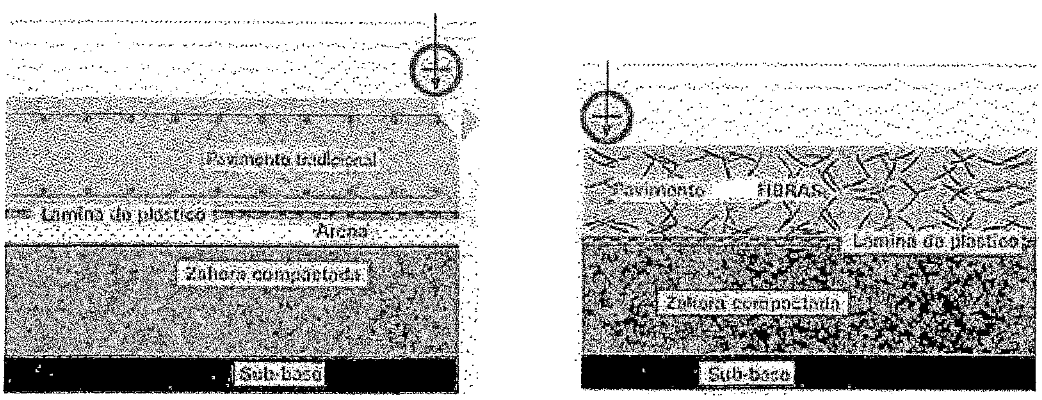
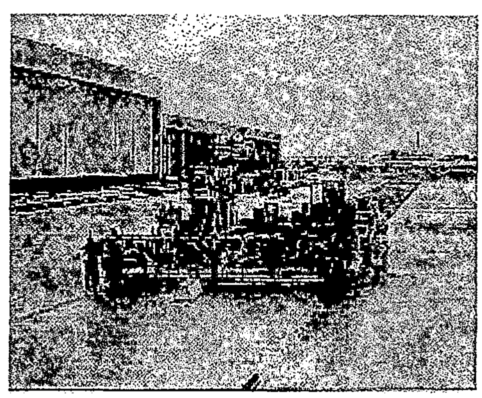


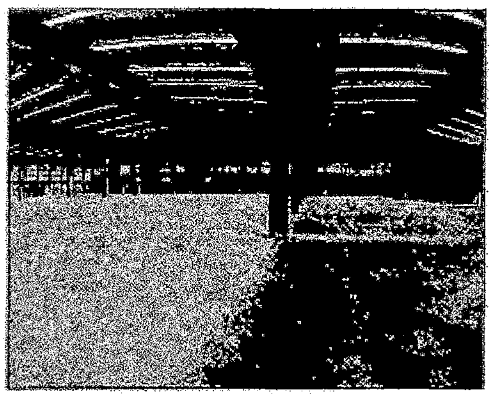
FIG. 4.9 APLICACIONES ADICIONALES

CÁMARA FRIGORIFICA



ESTACIONAMIENTO DE AVIONES

LOSA DE CIMENTACIÓN



PISOS INDUSTRIALES

CAPÍTULO 5

CONCRETO

5.1 GENERALIDADES

El concreto es básicamente una mezcla de dos componentes: Agregado y pasta. La pasta, compuesta de Cemento Pórtland y agua, une a los agregados (arena y grava o piedra triturada) para formar una masa semejante a una roca pues la pasta endurece debido a la reacción química entre el Cemento y el agua. Hoy en día esta definición abarca una amplia gama de productos; hay concretos hechos con diferentes tipos de cementos: puzolana, ceniza, escoria de alto horno, aditivo, sulfuro, ingredientes para mezcla, polímeros y fibras, entre otros.

Los agregados generalmente se dividen en dos grupos: finos y gruesos. Los agregados finos consisten en arenas naturales o manufacturadas con tamaños de partícula que pueden llegar hasta 10mm; los agregados gruesos son aquellos cuyas partículas se retienen en la malla No. 16 y pueden variar hasta 152 mm. El tamaño máximo de agregado que se emplea comúnmente es el de 19 mm o el de 25 mm.

La pasta esta compuesta de Cemento Pórtland, agua y aire atrapado o aire incluido intencionalmente. Ordinariamente, la pasta constituye del 25 al 40 % del volumen total del concreto. El volumen absoluto del Cemento esta comprendido usualmente entre el 7 y el 15 % y el agua entre el 14 y el 21 %. El contenido de aire y concreto con aire incluido puede llegar hasta el 8% del volumen del concreto, dependiendo del tamaño máximo del agregado grueso.

Como los agregados constituyen aproximadamente el 60 al 75 % del volumen total del concreto, su selección es importante. Los agregados deben consistir en partículas con resistencia adecuada así como resistencias a condiciones de exposición a la intemperie y no deben contener materiales que pudieran causar deterioro del concreto. Para tener un uso eficiente de la pasta de cemento y agua, es deseable contar con una granulometría continua de tamaños de partículas.

La calidad del concreto depende en gran medida de la calidad de la pasta. En un concreto elaborado adecuadamente, cada partícula de agregado esta completamente cubierta con pasta y también todos los espacios entre partículas de agregado.

5.2 VENTAJAS Y PROPIEDADES DEL CONCRETO

Para cualquier conjunto especifico de materiales y de condiciones de curado, la cantidad de concreto endurecido esta determinada por la cantidad de agua utilizada en la relación con la cantidad de Cemento. A continuación se presentan algunas ventajas que se obtienen al reducir el contenido de agua:

a) Se incrementa la resistencia a la compresión y a la flexión.
b) Se tiene menor permeabilidad, y por ende mayor hermeticidad y menor absorción.
c) Se incrementa la resistencia al intemperismo.
d) Se logra una mejor unión entre capas sucesivas y entre el concreto y el esfuerzo.
e) Se reducen las tendencias de agregamientos por contracción.

Entre menos agua se utilice, se tendrá una mejor calidad de concreto – a condición que se pueda consolidar adecuadamente. Menores cantidades de agua de mezclado resultan en mezclas más rígidas; pero con vibración, a un las mezclas más rígidas pueden ser empleadas. Para una calidad dada de concreto, las mezclas más rígidas son las más económicas. Por lo tanto, la consolidación del concreto por vibración permite una mejora en la calidad del concreto y en la economía.

Las propiedades del concreto en estado fresco (plástico) y endurecido, se pueden modificar agregando aditivos al concreto, usualmente en forma liquida, durante su dosificación. Los aditivos se usan comúnmente para:

a) Ajustar el tiempo de fraguado o endurecimiento
b) Reducir la demanda de agua
c) Aumentar la trabajabilidad
d) Incluir intencionalmente aire
e) Ajustar otras propiedades del concreto.

Después de un proporcionamiento adecuado, así como, dosificación, mezclado, colocación, consolidación, acabado, y curado, el concreto endurecido se transforma en un material de construcción resistente, no combustible, durable, resistencia al desgaste y prácticamente impermeable que requiere poco o nulo mantenimiento. El concreto también es un excelente material de construcción porque puede moldearse en una gran variedad de formas, colores y texturizados para ser usado en un numero ilimitado de aplicaciones

5.3 MEZCLA COLOCACIÓN Y COMPACTACIÓN DEL CONCRETO

5.3.1 MEZCLADO

Esta operación de mezclado consiste en la rotación o batido, con el propósito de cubrir la superficie de todas las partículas del agregado con la pasta de cemento y mezclar todos los ingredientes del concreto en una masa uniforme; esta uniformidad no debe afectarse durante el proceso de descarga de la mezcladora.

El tamaño de una mezcladora se define por el volumen de concreto que produce después de la compactación, diferente del volumen en los ingredientes no mezclados en un estado suelto, el cual es hasta 50% mayor que el volumen compactado. Las mezcladoras se fabrican en una variedad de tamaños desde 0.04 m^3 para uso de laboratorio hasta de 13 m^3. Si la cantidad mezclada representa solo una pequeña fracción de la capacidad de la mezcladora, la operación resultará antieconómica y la mezcla resultante puede no quedar uniforme, por lo que esta es una mala práctica. Sobrecargar la mezcladora hasta 10% más, generalmente es perjudicial, pero si se sobrepasa no se logrará una mezcla uniforme.

No hay un orden en que se alimenten loą ingredientes dentro de la mezcladora, ya que dependen de las propiedades de esta y de la mezcla. Generalmente se introduce una pequeña cantidad de agua, seguida de todos los demás sólidos, de preferencia alimentados de manera uniforme y simultánea. La mayor parte del agua deberá introducirse al mismo tiempo, dejando el resto para agregar después de los sólidos.

Para realizar este trabajo de investigación se procedió en el orden siguiente: Primeramente se introdujo una pequeña cantidad de agua debido a que la mezcladora absorbe y no afecte el agua de diseño, posteriormente se adiciono el agregado grueso y agregado fino dejando que se mezclen por unos segundos, luego en pleno movimiento de la mezcladora esparcir la fibra evitando que estas se junten, para que exista uniformidad en la masa de concreto, luego se adiciona una cierta cantidad del agua de diseño y posteriormente el cemento, el resto del agua se ira añadiendo hasta lograr que la masa de concreto tenga aceptación para encontrar un valor en el rango de la trabajabilidad con la medida del asentamiento.

El tiempo óptimo de mezclado depende del tipo y tamaño de la mezcladora, de la velocidad de rotación y de la calidad de los materiales al cargar la mezcladora. Por lo general un tiempo de mezclado de menos de 1 1 ¼ min produce una no uniformidad en la composición y una resistencia menor; el mezclado por mas de 2 min no causa una mejoría significativa en estas propiedades.

En la tabla 5.1 se proporcionan los valores típicos de tiempo de mezclados para mezcladoras de diferentes capacidades. El tiempo de mezclado se calcula desde el momento en que todos los materiales sólidos han sido ingresados en la mezcladora; el agua debe agregarse a mas tardar, después de un cuarto del tiempo total de mezclado.

TABLA 5.1

CAPACIDAD DE LA MEZCLADORA (m^3)	TIEMPO DE MEZCLADO (min)
0.8	1
1.5	1 1/4
2.3	1 1/2
3.1	1 3/4
3.8	2
4.6	2 1/4
7.6	3 1/4

5.3.2 COLOCACIÓN Y COMPACTACIÓN

La colocación y compactación son operaciones interdependientes y se llevan a cabo en forma casi simultanea. Son de gran importancia por el propósito de asegurar los requerimientos de resistencia, impermeabilidad y durabilidad del concreto endurecido.

El objetivo de la colocación es depositar el concreto tan cerca de su posición final, evitando la segregación y permitiendo su compactación completa.

El propósito de la compactación es remover todo el aire atrapado que sea posible, para que el concreto endurecido tenga un mínimo de vacíos. El concreto de revenimiento bajo contiene mas aire atrapado que el de revenimiento alto y por lo tanto el primero requiere mas esfuerzo para compactarse satisfactoriamente. Este esfuerzo lo provee principalmente el uso de vibradores, cuyo proceso consiste básicamente, en la eliminación de aire atrapado y en forzar a las partículas a una configuración mas estrecha. Las mezclas muy secas y espesas pueden hacerse vibrar satisfactoriamente y, en comparación con la compactación manual, alcanzan la resistencia deseada dada con un menor contenido de cemento. Esto significa ahorro en costo, aunque debemos compensar el costo del equipo de vibración y de un encofrado más pesado y resistente De cualquier modo el costo de mano de obra probablemente será el factor decisivo, en lo que a costos se refiere.

Tanto la compactación manual como por vibrador pueden producir concreto de buena calidad, contando con mezcla y mano de obra adecuadas. Igualmente con ambos métodos pueden producir mal concreto: en el concreto de apisonado a mano, la mano compactación es un error común, mientras que en caso de una vibración incorrecta o sobrevibración, se dará una compactación no uniforme y se ocasionará separación; esta última se puede evitar empleando una mezcla espesa y bien graduada.

La consistencia especificada de la mezcla determinará la elección del vibrador. Para una compactación eficiente, la consistencia del concreto y las características del vibrador disponible deben corresponderse mutuamente.

5.4 CONCRETO LANZADO

5.4.1 GENERALIDADES

El concreto con agregado fino es conocido como GUNITE y cuando incluye agregado grueso se designa como SHOTCRETE.

El Shotcrete usando la fibra de acero fue primero practicado en Norteamérica a comienzos de 1971 en un trabajo experimental bajo la dirección de D. R. Lankard . demostró que la fibra de acero era muy compatible con el concreto lanzado ya que mejora su resistencia, ductilidad y su capacidad de carga, aunque su uso indiscriminado genera concreto lanzado frágil debido a que esta ocasiona problemas de bloqueo. Su forma y dimensiones han cambiado considerablemente pero las más usuales son de 25 a 40 mm de longitud, incrementando el peso en rangos de 30 a 80 kg/m^3. Ensayos adicionales fueron realizados bajo la dirección de M. E. Poad por la Oficina de Minas de los U. S. en una investigación de nuevo y mejorado método para el uso del Shotcrete para soportes subterráneos. Subsecuentemente, R. A. Kaden de la Corporación de Ingenieros de la U. S. supervisó la primera aplicación practica de fibra de acero en un túnel en Ririe Dam, Idaho, en 1973. Desde ese tiempo, el concreto reforzado con fibra de acero ha sido puesto en practica en Alemania, Suiza, Inglaterra, Noruega, Finlandia, Holanda, Polonia, África del Sur, Australia, Canadá y Japón

El Gunitado, utilizando fibras de acero inoxidable, es una técnica frecuente actualmente en los revestimientos refractarios de industrias metalúrgicas, de cementos y petroquímicos. El utilizar fibras de inoxidables tiene como finalidad evitar las fuertes corrosiones que se producirían en fibras normales, al estar sometidos a atmósferas tan agresivas como las que suelen existir en los hornos.

En la actualidad se estima que el Gunite se utiliza en el 45% de los casos y el Shotcrete en el 55%, teniendo entre ambos una producción estimada de 8 millones de m3 por año en todo el mundo, la cual está en constante crecimiento.

La más reconocida definición de CONCRETO LANZADO (Shotcrete, Gunite) es la señalada por el AMERICAN CONCRETE INSTITUTE (ACI). Ellos han tomado el liderazgo al establecer comités técnicos así como especificaciones y publicaciones sobre esta materia. (ACI 506 de 1983 con actualizaciones en 1990 y 1994). "CONCRETO LANZADO es un mortero o concreto transportado a través de una manguera y proyectado neumáticamente a alta velocidad sobre una superficie".

Pudiendo ser dicha superficie: concreto, piedra, terreno natural, mampostería, acero, madera, poliestireno, etc.

Se considera que si la mezcla a lanzar cuenta sólo con agregados finos se llama Mortero Lanzado y si los agregados son gruesos se denomina Concreto.

Hay dos clasificaciones de concreto lanzado: seco (al que se le añade el agua en la boquilla) y húmedo (al que el agua se le añade antes de entrar por la manguera).

El concreto conducido a través de tubería de acero y que no es proyectado ni transportado a altas velocidades se conoce como concreto bombeado.

5.4.2 DIFERENCIA DEL CONCRETO LANZADO Y EL CONCRETO TRADICIONAL

A diferencia del concreto convencional, el cual es colocado y luego compactado (vibrado) en una segunda operación, el Concreto Lanzado es colocado y compactado al mismo tiempo, debido a la fuerza con que es proyectado desde la boquilla.

Ventajas del Concreto Lanzado sobre el Concreto Tradicional

a) Diseño de formas libres.
b) Eliminación de cimbras y de tiras de corte.
c) Alta calidad de concreto: Alto contenido de cemento, baja relación agua-cemento, concreto denso de alta resistencia, baja permeabilidad, ideal para estructuras de pared delgada.
d) Construcción eficiente en grandes áreas con paredes esbeltas.
e) Terminados agradables.
f) Acceso a sitios difíciles ya que se pueden alcanzar 300 m. horizontales y 100 m. verticales.
g) Disminución de grietas por temperatura.
h) Facilidad de unión entre diferentes capas.

5.4.3 TIPOS DE CONCRETO LANZADO

5.4.3.1 CONCRETO LANZADO EN SECO

Equipo

Existen equipos que realizan este proceso entre los más usados están:

Suministradora y mezcladora en tránsito, es usada solo para concretos sin aditivos u otros materiales. Batido en el lugar, es el más usado de los métodos de mezclado y suministro de mortero, a demás se reduce el tiempo entre la elaboración y el uso del mismo, aunque es el menos industrializado es el más eficiente por su versatilidad, pudiendo hacerse el proporcionamiento por peso o por volumen, se presentan en modalidades fija y móvil.

En este proceso se utilizan:

a) Una Lanzadora para concreto en seco.
b) Máquina con cámara de presión, máquina tipo tornillo, máquina tipo rotor y lanzadoras automáticas.
c) Una mezcladora de mortero.
d) Un compresor.
e) Una bomba de agua.
f) Una planta generadora de electricidad.

La boquilla en el proceso de lanzado en seco tiene un papel predominante ya que es la zona donde se impregna el mortero con el agua. La más común es un tubo de plástico con válvula reguladora del ingreso del agua, la boquilla durante el lanzado se mueve de forma pausada circularmente u elipsoidalmente, si el lanzador no cuenta con la experiencia adecuada redundara en el incremento del rebote.

Materiales

Los materiales usados en el concreto lanzado son básicamente los mismos que los utilizados en el concreto bombeable, cemento, agregados inertes finos, agua, aditivos y a partir de los 80s se empezaron a utilizar fibras de acero y polipropileno, ceniza volcánica, cementos especiales, humos de sílice, etc.

Materiales cementantes

Antes de los 80s todo el cemento usado era del tipo Pórtland, pero a partir de esta fecha se comenzaron a utilizar otros como el que contenía alúmina en aplicaciones refractarias, el puzolánico para condiciones con sales, etc.

Los agregados tienen que tener granulometría fina porque de lo contrarío se incrementaría el rebote y el taponamiento del equipo. Los agregados más utilizados son los de origen en ríos o marinos, el peso del mortero es muy bajo anda alrededor de 500 kg/m3. El tamaño máximo de los agregados es del orden de los 10 mm (3/8").

Materiales suplementarios a los cementantes

Humos de sílice es el desarrollo más importante en el concreto lanzado durante los 70's, siendo los noruegos los pioneros en el campo. Se utilizan en rangos de 7 al 15% en peso de cemento junto con aditivos reductores y superfluidizantes en 10% en peso.

Mediante su uso se generan los siguientes resultados en la mezcla:

Incremento de la adhesión y la cohesión, uso de espesores mayores sin necesidad de aditivo acelerante, incremento de resistencia a flexión y a compresión, disminuye el rebote, incremento de la resistencia a las sales y a los ataques químicos, incremento de la impermeabilidad, se utiliza por igual en ambos tipos de lanzado.

Aditivos

Acelerantes.- tradicionalmente los aditivos acelerantes han sido los más usados en los procesos en seco, en virtud de poder incrementar el espesor del lanzado.

Los inclusores de aire.- se usan en mezclas para mejorar la durabilidad, prácticamente no se usan en mezclas en seco, cuando ocasionalmente se usan se mezclan con el agua y se aplican en la boquilla aumentando la resistencia a los ataques químicos.

Fibra de refuerzo

Fibra de acero

Fibra de polipropileno

Persona

La distribución del personal se hará básicamente en dos cuadrillas, donde la primera se encargará de la elaboración del mortero así como el suministro del mismo en la lanzadora, la segunda en cambio se dedicara al lanzado propiamente dicho. El personal que conforma las cuadrillas es el siguiente: Un sobrestante, un lanzador, un asistente del lanzador, un operador del compresor, un operador de la compresora, un operador de la lanzadora, un operador de la mezcladora así como por varios peones

El sobrestante es el supervisor, organizador y encargado del buen funcionamiento de las cuadrillas en lo individual y del proceso en lo general, siendo capaz de la toma de ciertas decisiones evitando que los imprevistos generen retrasos sustanciales en los programas de obra preestablecidos, siendo a la vez el líder emocional del equipo.

El lanzador debe ser una persona con un grado de capacitación y sensibilidad en la tecnología del concreto, ya que esto nos redundará en la optimización de los procesos a realizar

El operador de la lanzadora es el responsable de proporcionar el mortero en las cantidades requeridas por el lanzador de forma oportuna permitiendo la eficiencia del proceso.

El asistente del lanzador debe vigilar el proceso de lanzado puntualizando oportunamente la existencia de algún elemento o situación anómala discordante del proceso previsto, siendo a su vez capaz de cubrir las labores del lanzador.

El operador del compresor debe tener un conocimiento tanto de su equipo como del resto de la maquinaría involucrada en el proceso de lanzado, permitiéndole ajustar su equipo a la frecuencia y rendimientos del resto, siendo esto parte fundamental del éxito de la obra a ejecutar. El operador de la mezcladora es el responsable de suministrar un mortero con el proporcionamiento y calidad solicitadas de manera oportuna y constante.

5.4.3.2 CONCRETO LANZADO EN HUMEDO

Mezclas en procesos en húmedo

Se requiere de un mortero con cemento, agregados finos y gruesos con un tamaño máximo de ¾", pudiendo incluir: aditivos, cenizas volcánicas, humos de sílice y fibras de acero y polipropileno. En este proceso la mezcla húmeda se procesa y se coloca en una bomba donde por medios hidráulicos se desliza hasta la boquilla donde se proyecta neumáticamente con velocidades desde los 10 hasta 30m/seg. Siendo posible el lanzado a 500 m horizontales y 150 m verticales

5.4.4 PROPIEDADES DEL CONCRETO LANZADO

Propiedades plásticas.

En pruebas con concretos húmedos reforzados se encontró que el mejor camino para incorporar fibras en las mezclas es con altos volúmenes de aire 20%, generando poco rebote 0.2% en muros y 0.5% en losas.

Resistencia a la compresión.

F'c= 140 a 315 kg/cm2. Esta resistencia depende en gran medida de la relación agua-cemento, las fibras la incrementan de un 5 a un 21%. Otro factor que influye mucho en la resistencia es el tiempo el cual llega a sus puntos críticos a los 7 días donde alcanza el 75% de la resistencia final y a los 28 días donde alcanza finalmente. Si se requiere variación en estos tiempos se utilizan los aditivos.

Resistencia a la flexión

Siempre se toma como una medida indirecta de la resistencia a la tensión del concreto, con valores de 21 a 35 kg/cm2 siendo los valores más altos entre 5 y 10 cm espesor. Si se utilizan relaciones agua-cemento bajas se alcanza hasta 56 kg/cm2 y con fibras se alcanzan los 70 kg/cm2 que ya es un valor significativo, aunque para cuestiones

Dureza flexionante

Siempre es asociada a la adición de fibra y es probada por medio de vigas de acero recubiertas por concreto rico en fibra, dando como resultados valor máximo de 42 kg/cm2.

Resistencia al impacto

También se incremento por medio de fibras metálicas hasta en un 13% dependiendo del espesor.

Unión de substratos

En procesos húmedos se agregan rocas como el granito y sienita variando su fuerza de unión hasta un máximo de 8.4 kg/cm2. Este proceso se utiliza en túneles. Ya que después de 24 horas se alcanza una resistencia del concreto de hasta 80%. A demás si se quiere incrementar esta se puede usar fibra de acero en procesos de mezcla húmeda con granito.

Tenacidad

Se incrementa al incorporar fibra al concreto lanzado, a demás le da ductilidad, absorción de energía, resistencia al impacto y a las grietas. Existen muchos estudios sobre las fibras y el porcentaje real de tenacidad que añaden a la mezcla y el resultado siempre depende del tipo de fibra por lo que se debe particularizar para cada caso en su uso según necesidades específicas.

Permeabilidad

No se puede considerar impermeable ni siquiera el concreto lanzado aunque tienda a serlo. Se ha encontrado que el uso de humos de sílice incrementa considerablemente esta capacidad en el concreto haciéndolo prácticamente impermeable y esto se prueba con su uso en estructuras marinas nuevas y la reparación de las mismas. En rangos de 0.37 a 0.41 m/m2, al agregar humos de sílice se obtienen rangos de 3X10-10 a 3X10-13. Se consideran impermeables desde 1x10-12.

Resistencia a la corrosión de las fibras de acero para concreto lanzado.

Se ha probado que las fibras de buena calidad no dan muchos problemas si son usadas debidamente, incluso estando en contacto con contaminación, ambiente marino, o ciertos químicos tardan años en dañarse y sólo se muestran los daños en concretos con grietas considerables donde la corrosión de las fibras aparecerá pronto.

Durabilidad

Siempre es un valor resultado de exponer una muestra de concreto al congelamiento y descongelamiento de la misma de manera sucesiva, la durabilidad del concreto en cuanto este es atacado por agentes químicos en caso de no contener los aditivos necesarios es muy poca, pero siempre será mayor que la del concreto convencional.

Grietas de temperatura y encogimiento

Estas deformaciones son mayores en mezclas en húmedo, pero incluso esta modalidad tiene mayor capacidad de reducir las deformaciones por esta vía que el concreto tradicional, las fibras metálicas reducen la formación de grietas y las de polipropileno el encogimiento por temperatura aunque no deben utilizarse de manera combinada.

Absorción de agua

Esta se incrementa hasta llegar a rangos de 3.5 a 4.7% para fibras metálicas y de 2.6 a 4.0% con humos de sílice.

Resistencia al congelamiento y descongelamiento

Siempre son más altas en las mezclas en seco debido a la relación agua-cemento muy baja y es alrededor del 8% mayor que en mezclas húmedas.

Corrosión del refuerzo

Depende del espesor, del tipo de concreto y el espesor en contacto con el agua (siempre mayor a 5 *cm.).*

Densidad

Puesto que está colocado bajo presión de aire a alta velocidad el Concreto Lanzado tiene una densidad de 2240 a 2400 kg./m3 con un mínimo de aire incluido.

Adherencia

El Concreto Lanzado pega perfectamente con superficies limpias y preparadas de concreto, ladrillo, cerámica, piedra y acero siendo más fuertes que el propio material donde fue aplicado, ya que aplicando una fuerza al corte, finalmente fallan los otros elementos.

Baja relación agua-cemento

Las excelentes propiedades físicas del material son debidas a que el agua que contiene es solo necesaria para satisfacer la hidratación química del cemento.

Resistencia a la abrasión

La resistencia a la abrasión se observó en una prueba realizada a 6 especimenes de Concreto Lanzado, 6 de Mortero colocado a mano y 6 de Concreto convencional con la misma proporción de mezclas, el resultado de desgaste fue el siguiente: Concreto Lanzado 7%, Mortero 16% y Concreto 11%.

5.4.4 USOS DEL CONCRETO LANZADO

Las primeras aplicaciones del Concreto Lanzado en estructuras nuevas fue en construcciones de formas libres, pero con el advenimiento de fibras, nuevos aditivos y agregados su uso se volvió muy amplio, considerando que es un concreto estructural que puede alcanzar la resistencia requerida y permitir cualquier tipo de acabado. Su uso tiene como límite sólo la imaginación, algunos ejemplos de su aplicación son:

a) Estructuras de tres dimensiones: domos geodésicos, paraboloides hiperbólicas, paraboloides, conos, cúpulas, silos.

b) Contenedores de líquidos: albercas, cisternas, tanques, canales.

c) Usos especiales: estructuras refractarias, estructuras resistentes al intemperismo.

d) Recubrimientos: en panel, acero, mampostería, concreto.

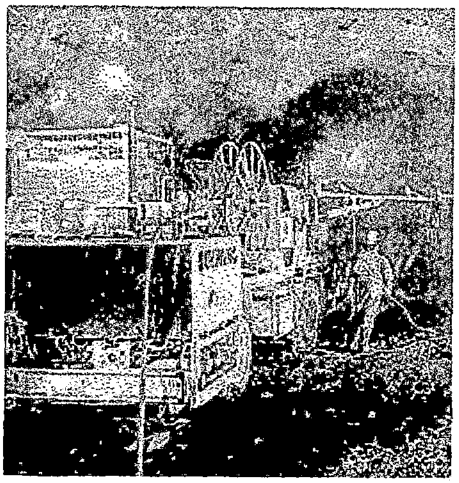
e) Estructuras de retención: túneles, estabilización de taludes, muros de retención.

f) Reparación de estructuras: Existen estructuras que han sido dañadas por un sismo, por el fuego, ataque de agentes químicos o bien sub-calculadas, mediante el Concreto Lanzado pueden ser corregidas. Algunos ejemplos son:
Estructuras dañadas por sismo o deficiencia de cálculo como: edificios, puentes.
Estructuras atacadas por agentes químicos como: muelles, tanques, canales.

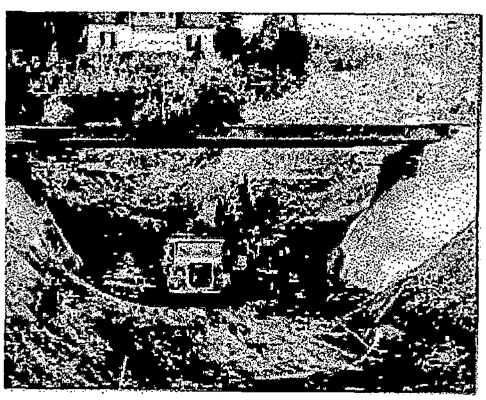
El concreto lanzado reforzado con fibras de acero constituye una solución durable, dúctil y económica para este tipo de aplicaciones. La eliminación de la malla como refuerzo en la estabilización de taludes, lineamientos para túneles, el proceso de clavado en terreno, rehabilitaciones y restauraciones, así como otras aplicaciones de esta especie, resulta en un decremento en costos y una disminución de desperdicio obteniendo una mejor calidad de concreto aplicado a un rango mayor.

En la figura 5.1 se muestra las aplicaciones del concreto lanzado.

FIG. 5.1 APLICACIONES DEL CONCRETO LANZADO

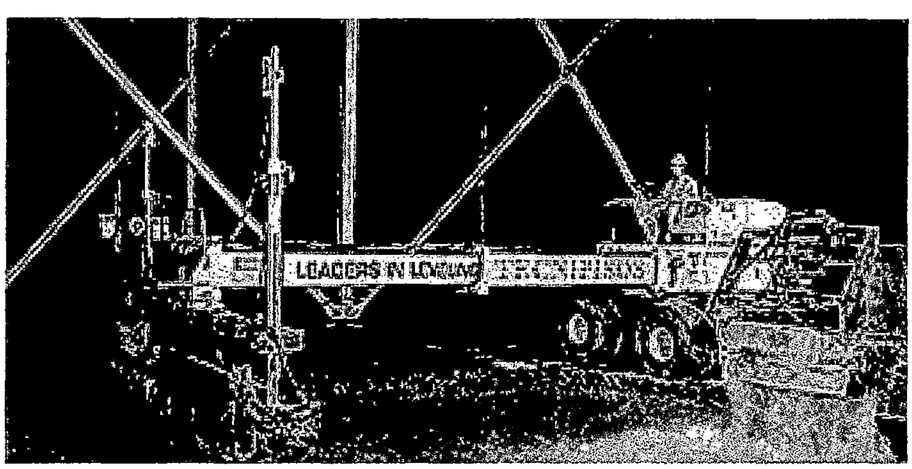


Concreto Lanzado en Túneles
Fig (a)



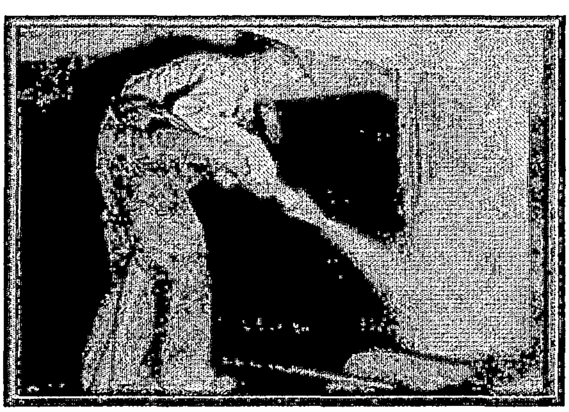
Concreto Lanzado en Estabilización de Taludes
Fig (b)

Concreto Lanzado en Pisos Industriales
Fig (c)



Concreto Lanzado en Columnas
Fig (d)

Concreto Lanzado en Muros de Contención
Fig (e)



Concreto Lanzado Para Rehabilitación
Fig (f)

CAPÍTULO 6

DISEÑO DE MEZCLA

6.1 GENERALIDADES

Se entiende como diseño de mezclas a la selección mas adecuada y conveniente de sus componentes como son: agua, cemento y agregados fino y grueso. Estos componentes de la mezcla se dosifican de manera que, el producto resultante llamado concreto tenga las características y/o requisitos establecidos, tanto en el estado fresco como en estado endurecido.

a) El concreto en estado fresco debe ser trabajable de tal modo que sea posible su colocación y

b) En el estado endurecido debe tener como características principales; la resistencia y la durabilidad adecuadas al propósito que se desea obtener, además de cumplir con las especificaciones técnicas requeridas.

Es importante, que después de que se han elegido los materiales para el concreto y determinado las dosificaciones su uso debe controlarse cuidadosamente. Este control en el campo determina la calidad, uniformidad y economía de la obra. Cuanto menor es la calidad de los ingredientes, mayor es la necesidad de plantear un control mas rígido para alcanzar una durabilidad satisfactoria y una resistencia uniforme, con lo cual alcanzaremos a darle la máxima vida útil a la estructura.

También existe otro factor importante que es el económico, por lo que la mezcla del concreto ha elaborarse debe ser de calidad con el menor costo posible conjuntamente a las condiciones principales antes mencionadas.

Hoy en día, existen varios métodos para lograr un diseño de mezclas de concreto, ningún método es perfecto, su aplicación dependerá muchas veces de los logros y experiencia que el profesional ha obtenido con ellos siempre y cuando no estuviera obligado a optar por uno de ellos, de acuerdo a las especificaciones técnicas del proyecto.

6.2 FACTORES QUE DEBEN CONSIDERARSE

RELACIÓN AGUA/CEMENTO

Para los efectos de estimar las relaciones agua/cemento nos remitimos a las tablas elaboradas por el ACI-211.1-91, aunque pese a no ser aplicables a nuestra realidad, nos permite encontrar un punto de inicio conservador y respaldado científicamente, posteriormente perfeccionaremos este parámetro sobre las bases de las mezclas de prueba y los resultados prácticos.

PROPORCION MINIMA DE CEMENTO

En muchas especificaciones se fija una proporción mínima de cemento. En esta forma se da protección contra los efectos del aumento en la demanda de agua debido al aumento de temperatura, o a los cambios de granulometría en el agregado fino, asegurándose así contra las bajas resistencias. Pueden influir en las proporciones mínimas de cemento las características de los materiales locales, las condiciones climáticas, la manera en que se haga el vaciado y la posterior exposición.

AIRE INCLUIDO

En todos los concretos que vayan a quedar expuestos a la congelación y fusión deberá usarse aire incluido y puede usarse en condiciones donde la exposición no sea muy intensa, para mejorar su manejabilidad. La cantidad de aire para producir la resistencia adecuada al congelamiento y la fusión depende del tamaño máximo del agregado.

Cuando se mantiene constante el agua en la mezcla, con la inclusión de aire se aumenta el revenimiento. Cuando la proporción de cemento y revenimiento de mantiene constantes, se requiere menos agua en la mezcla; la disminución que resulta en la relación agua/cemento ayuda a contrarrestar posibles disminuciones en la resistencia, originando mejoras en otras propiedades de la pasta, como en la permeabilidad.

SELECCIÓN DEL AGREGADO

En el concreto reforzado, el tamaño máximo del agregado que se puede usar depende del ancho de la sección y del espaciamiento del refuerzo, debemos recordar que si bien las propiedades del concreto mejoran al aumentar el tamaño del agregado, este no debe ser mayor de 40 mm (1 ½"), por lo que el uso de tamaños mas grandes no es recomendable.

La cantidad necesaria del agua de mezcla para producir un metro cúbico de concreto de un revenimiento determinado depende del tamaño máximo del agregado ; cuanto menor sea el tamaño máximo del agregado, mayor será la cantidad de agua. Se recomienda usar el mayor tamaño que sea practico en el agregado grueso.

REVENIMIENTO O ASENTAMIENTO

Generalmente se usa la prueba de revenimiento o asentamiento como medida de la consistencia del concreto. No debe usarse para comparar mezclas de proporciones completamente diferentes, ni mezclas con diferente clase o tamaños de agregados. Cuando se usa en diferentes muestras de la misma mezcla; los cambios en el revenimiento indicaran cambios en los materiales, en las proporciones de la mezcla, o en la cantidad de agua.

6.3 CRITERIO DE DISEÑO

En el presente trabajo el diseño de mezclas se basa en el criterio de optimización de los materiales, especialmente del cemento, por ser el de mayor costo, de modo que la mezcla ha elaborarse sea económica. Y para conseguirlo es indispensable realizar una buena combinación de agregados de tal manera que dada una relación agua / cemento y dado un asentamiento, obtener mediante dicha combinación una mezcla de concreto con la mínima dosificación de cemento y/o máxima resistencia a la compresión en el concreto endurecido.

6.4 COMBINACIÓN DE AGREGADOS CON MAYOR PESO UNITARIO

El cual consiste en obtener una relación de agregados que nos permita una optima mezcla de concreto. Para ello se debe hallar la relación que nos proporcione una combinación de agregados con máximo peso unitario. Esto es con mínima relación de vacíos, se asegura de este modo la condición de economía, mas no así las propiedades de resistencia y durabilidad, del que posteriormente se analizara un análisis al diseñar la mezcla, variando la relación porcentual evaluando puntos, debajo y encima de alrededor del mayor valor que se haya obtenido para una optima relación de agregados, hasta obtener así la máxima resistencia a la compresión.

En el capitulo 2 cuadro 2.13, se muestra el cuadro de resultados de los diversos porcentajes de agregados con sus respectivos pesos unitarios compactados del agregado global.

En la grafica 2.2, se puede observar que para 50% de arena y 50% de piedra (porcentaje en peso), se obtiene el mayor peso unitario compactado. Pero, para una mejor determinación del agregado global, se hará el ensayo de la resistencia a la compresión de 3 probetas a la edad de 7 días, con un asentamiento de 3 a 4 pulg. Con el porcentaje de 50% de arena (máximo peso unitario) y 2 puntos a ± 3% debajo y encima del máximo obtenido de dicha combinación, de las cuales se utilizara para el diseño el punto de mayor resistencia a la compresión. (ver grafico 2.3 del capítulo 2)

6.5 PROPIEDADES FÍSICAS DE LOS MATERIALES A EMPLEAR

Debemos tener en cuenta al momento de dosificar cualquier mezcla de concreto, las propiedades físicas de los materiales; así como la combinación de agregados de con mayor peso unitario compactado que nos asegure la mayor densidad de mezcla o una mínima proporción de vacíos.

Los datos de los materiales empleados en el diseño de mezcla son los siguientes:

CEMENTO POTLAND TIPO I ANDINO

PESO ESPECIFICO 3.12 gr/cm3

CUADRO N° 6.1

RESUMEN DE LAS CARACTERISTICAS FISICAS

Descripcion	Und	Agregado	
		Fino	Grueso
Peso Específico de Masa	gr/cc	2.643	2.750
Porcentaje de Absorción	%	2.460	0.469
Contenido de Humedad	%	1.490	0.267
Tamaño máximo Nominal	Plg	----	1"
Combinacion	%	51	49

6.6 SECUENCIA DE DISEÑO

A continuación se presenta la secuencia seguida en el diseño de mezcla:

a) Determinación de la propiedades físicas de los materiales a emplear.

b) Elección de la relación agua/cemento en peso. Si estuviéramos en obra se elegiría la relación agua/cemento sobre la base de la resistencia a la compresión requerida o condiciones de durabilidad.

c) Elección del revenimiento o asentamiento según la consistencia requerida ao las condiciones de trabajabilidad.

d) Se considera el tamaño nominal máximo del agregado grueso

e) Se determina si la mezcla tendrá o no aire incorporado. Se estima el porcentaje de aire por metro cúbico y el volumen absoluto que atrapara el concreto en función del Tamaño Nominal Máximo del agregado grueso.

f) Se establece la cantidad de agua por metro cúbico en función del Tamaño Nominal Máximo del agregado, del asentamiento y considerando si la mezcla tiene aire atrapado o incorporado. Esto se establece de las tablas del ACI.

g) Se calcula cantidad de cemento en peso, basándose en la relación agua/cemento y la cantidad de agua a emplear por metro cúbico de concreto.

$$\text{Cemento(Kg)} = \frac{\text{Peso del agua (Kg)}}{\text{Relacion agua/cemento}}$$

h) Calculo de los volúmenes absolutos del agua y cemento:

$$\text{Volumen Absoluto del Agua } (m^3) \quad \frac{\text{Peso del Agua (Kg)}}{\text{Peso Especifico Agua } (Kg/m^3)}$$

$$\text{Volumen Absoluto del Cemento } (m^3) \quad \frac{\text{Peso del Agua (Kg)}}{\text{Peso Especifico Agua } (Kg/m^3)}$$

i) Después de conocer los volúmenes que ocupan el agua, cemento y aire atrapado; se procede a calcular el volumen, que ocuparan los agregados para un metro cubico de concreto.

$$\text{Vol. Abs.Agreg}(m^3) = 1 - (\text{Vol.Cemento+Vol. Agua+Vol. Aire Atrapado})$$

j) Ahora se calcula el volumen de los agregados fino (vf) y grueso (vg) sabiendo que:

$$Vf + Vg = \text{Vol. Abs. Agregados} \quad \ldots\ldots\ldots\ldots\ldots \text{I}$$

$$\%\ \text{Ag.Fino} = \frac{\text{P.E. (Fino)} \times Vf}{\text{P.E. (Fino)} \times Vf + \text{P.E. (Grueso)} \times Vg} \quad \ldots\ldots\ldots\ldots\ldots \text{II}$$

Resolviendo I y II se hallan los volúmenes de los agregados fino (Vf) y (Vg).

k) Luego se calcula los pesos secos de los agregados:

Peso seco Arena (Kg) = Vol. Ag. Fino(m^3) x P.E. de la arena (Kg/ m^3)

Peso seco Piedra (Kg) = Vol. Ag. Grueso(m^3) x P.E. de la Piedra (Kg/ m^3)

l) Se continua calculando el aporte de agua de los agregados:

Agua de la Arena = Peso seco Arena x (% Cont. Humed. - %Absor.)/100

Agua de la Piedra = Peso seco Piedra x (% Cont. Humed. - %Absor.)/100

m) Corrección de la cantidad de agua

Agua de mezcla = Agua Inicial – (Agua de la Arena + Agua de la Piedra)

n) Calculo del Peso Húmedo de los Agregados:

Peso húmedo de la Arena = Peso seco Arena x (1+ %Cont. Humedad)

Peso húmedo de la Piedra = Peso seco Piedra x (1+ %Cont. Humedad)

o) Finalmente tendremos el diseño de mezcla para un metro cúbico de concreto:

Cemento (Kg); Agua(Lts); Peso Húmedo de la Piedra (Kg); Peso Húmedo de la Arena (Kg).

Con este diseño se obtuvo la dosificación para un metro cúbico de concreto; pero en la presente investigación se uso una mezcladora de 0.021 m^3

Siguiendo el método del ACI se realizo una mezcla de prueba en la que se obtuvo un concreto que no era trabajable (Asentamiento 1").

Se presenta en el cuadro N° 6.2 los diseños de mezclas para la relación agua/cemento = 0.65 variando las proporciones de agregados. Luego se hará ensayos de Resistencia a la Compresión como se muestra en el cuadro N° 2.15 (capitulo 2).

CUADRO N° 6.2

DISEÑO DE MEZCLAS

MEZCLA	MATERIAL	PESO (KG/M^3)	VOL ABS. (M^3)	TANDA 54 kg
A/C=0.65 %Ag.Fino 47	Cemento	320.00	0.10	7.28
	Agua	208.00	0.21	4.97
	Ag. Fino	854.97	0.32	19.75
	Ag. Grueso	964.11	0.35	22.00
	Σ Total	2347.08	0.98	54.00
	%Aire de Diseño		1.50%	
	Asentamiento	4 1/2"		
A/C=0.65 %Ag.Fino 50	Cemento	327.69	0.11	7.49
	Agua	213.00	0.21	5.11
	Ag. Fino	898.37	0.34	20.83
	Ag. Grueso	898.37	0.33	20.58
	Σ Total	2337.43	0.98	54.00
	%Aire de Diseño		1.50%	
	Asentamiento	5 1/2"		
A/C=0.65 %Ag.Fino 53	Cemento	330.77	0.11	7.57
	Agua	215.00	0.22	5.17
	Ag. Fino	946.85	0.35	21.99
	Ag. Grueso	839.66	0.30	19.27
	Σ Total	2332.27		54.00
	%Aire de Diseño		1.50%	
	Asentamiento	5 1/2"		

Con los datos obtenidos se procede a realizar la curva de ensayo de resistencia a la compresión a los 07 días como muestra la grafica N° 2.3 (capítulo 2), donde observamos que la relación de 51% de arena y 49% de piedra, tiene mejor resistencia que las otras tres., además nos proporciona una mejor trabajabilidad del concreto.

Por lo tanto, esta es la relación optima que se utilizara para el diseño de mezclas.

6.7 DISEÑO DE MEZCLAS PARA LAS RELACIONES A/C = 0.60, 0.65, 0.70

Para el presente tema de investigación, se ha diseñado relaciones agua/ cemento: 0.60, 0.65, 0.70, mantendremos constante el porcentaje de agregados óptimos hallado (% arena = 51%), para los 3 diseños del presente trabajo de investigación, obtendremos de esta manera un concreto de máxima resistencia, buena trabajabilidad con una adecuada consistencia.

Para el diseño de mezclas, se utiliza el método de dosificación del A.C.I. 211.1.81 referente al principio de volúmenes absolutos; de tablas se estima, se estima la primera aproximación de la cantidad de agua (aunque dichos cuadros están elaborados para cementos normales, para nuestros ensayos este valor nos resulta muy por debajo del requerido para el cemento Pórtland tipo I Andino, mediante ajustes sucesivos de su curva correspondiente se determina la cantidad de agua necesaria para conseguir un asentamiento de 4 ½" a 5 ½"; este proceso se realiza para cada relación agua – cemento. Posteriormente se calcula la cantidad de piedra y arena para un metro cúbico de concreto, en base a los porcentajes de arena obtenidos. Cabe señalar que para el diseño optimo no solo se ha encontrado el valor de porcentaje de arena, sino también el volumen compactado de agregados grueso optimo por metro cúbico de concreto.

6.8 DOSIFICACIÓN DEL CONCRETO NORMAL PARA LOS ENSAYOS

En el cuadro N° 6.3 que se adjunta, se muestra la dosificación para cada una de las relaciones de agua/cemento con Agregado Fino = 51% optimo.

Para la determinación de la cantidad de agua necesaria y proceso de ensayo se puede ver en el anexo B.

CUADRO N° 6.3

DISEÑO DE MEZCLAS FINALES

RELACION (A/C)	MATERIAL	PESO (KG/M3)	VOL ABS. (M3)	TANDA 50 kg
0.60	Cemento	358.33	0.11	7.58
	Agua	225.48	0.22	4.77
	Ag. Fino	913.13	0.34	19.32
	Ag. Grueso	866.75	0.31	18.33
	Σ Total	2363.70	0.98	50
	%Aire de Diseño		1.50%	
	Asentamiento	4 1/2"		
0.65	Cemento	329.23	0.11	6.97
	Agua	224.65	0.21	4.76
	Ag. Fino	927.53	0.34	19.64
	Ag. Grueso	880.42	0.32	18.64
	Σ Total	2361.82	0.98	50
	%Aire de Diseño		1.50%	
	Asentamiento	4 1/2"		
0.70	Cemento	304.29	0.10	6.45
	Agua	223.79	0.21	4.74
	Ag. Fino	940.07	0.35	19.91
	Ag. Grueso	892.32	0.32	18.90
	Σ Total	2360.46	0.98	50
	%Aire de Diseño		1.50%	
	Asentamiento	5"		

6.9 DOSIFICACION DEL CONCRETO CON FIBRAS PARA LOS ENSAYOS

6.9.1 DOSIFICACION DE FIBRA: 35 Kg/m^3 DE CONCRETO

CUADRO N° 6.3

DISEÑO DE MEZCLAS FINALES

RELACIÓN (A/C)	MATERIAL	PESO (KG/M3)	VOL ABS. (M3)	TANDA 50 kg
0.60	Cemento	358.33	0.11	7.58
	Agua	225.48	0.22	4.77
	Ag. Fino	913.13	0.34	19.32
	Ag. Grueso	866.75	0.31	18.33
	Σ Total	2363.70	0.98	50
	FIBRA	35.00		0.740
	%Aire de Diseño		1.50%	
	Asentamiento	3 1/2"		
0.65	Cemento	329.23	0.11	6.97
	Agua	224.65	0.21	4.76
	Ag. Fino	927.53	0.34	19.64
	Ag. Grueso	880.42	0.32	18.64
	Σ Total	2361.82	0.98	50
	FIBRA	35.00		0.741
	%Aire de Diseño		1.50%	
	Asentamiento	3 1/2"		
0.70	Cemento	304.29	0.10	6.45
	Agua	223.79	0.21	4.74
	Ag. Fino	940.07	0.35	19.91
	Ag. Grueso	892.32	0.32	18.90
	Σ Total	2360.46	0.98	50
	FIBRA	35.00		0.742
	%Aire de Diseño		1.50%	
	Asentamiento	3"		

6.9.2 DOSIFICACION DE FIBRA: 45 Kg/m^3 DE CONCRETO

CUADRO N° 6.4

DISEÑO DE MEZCLAS FINALES

RELACION (A/C)	MATERIAL	PESO (KG/M3)	VOL ABS. (M3)	TANDA 50 kg
0.60	Cemento	358.33	0.11	7.58
	Agua	225.48	0.22	4.77
	Ag. Fino	913.13	0.34	19.32
	Ag. Grueso	866.75	0.31	18.33
	Σ Total	2363.70	0.98	50
	FIBRA	45.00		0.952
	%Aire de Diseño		1.50%	
	Asentamiento	3 3/4"		
0.65	Cemento	329.23	0.11	6.97
	Agua	224.65	0.21	4.76
	Ag. Fino	927.53	0.34	19.64
	Ag. Grueso	880.42	0.32	18.64
	Σ Total	2361.82	0.98	50
	FIBRA	45.00		0.953
	%Aire de Diseño		1.50%	
	Asentamiento	3 1/2"		
0.70	Cemento	304.29	0.10	6.45
	Agua	223.79	0.21	4.74
	Ag. Fino	940.07	0.35	19.91
	Ag. Grueso	892.32	0.32	18.90
	Σ Total	2360.46	0.98	50
	FIBRA	45.00		0.954
	%Aire de Diseño		1.50%	
	Asentamiento	3 1/2"		

6.9.3 DOSIFICACION DE FIBRA: 55 Kg/m^3 DE CONCRETO

CUADRO N° 6.5

DISEÑO DE MEZCLAS FINALES

RELACIÓN (A/C)	MATERIAL	PESO (KG/M3)	VOL ABS. (M3)	TANDA 50 kg
0.60	Cemento	358.33	0.11	7.58
	Agua	225.48	0.22	4.77
	Ag. Fino	913.13	0.34	19.32
	Ag. Grueso	866.75	0.31	18.33
	Σ Total	2363.70	0.98	50
	FIBRA	55.00		1.163
	%Aire de Diseño		1.50%	
	Asentamiento	3 1/8"		
0.65	Cemento	329.23	0.11	6.97
	Agua	224.65	0.21	4.76
	Ag. Fino	927.53	0.34	19.64
	Ag. Grueso	880.42	0.32	18.64
	Σ Total	2361.82	0.98	50
	FIBRA	55.00		1.164
	%Aire de Diseño		1.50%	
	Asentamiento	3 1/4"		
0.70	Cemento	304.29	0.10	6.45
	Agua	223.79	0.21	4.74
	Ag. Fino	940.07	0.35	19.91
	Ag. Grueso	892.32	0.32	18.90
	Σ Total	2360.46	0.98	50
	FIBRA	55.00		1.165
	%Aire de Diseño		1.50%	
	Asentamiento	3"		

CAPÍTULO 7

PROPIEDADES DEL CONCRETO

7.1 ENSAYOS EN EL CONCRETO FRESCO

Las propiedades del concreto fresco están en función a la utilidad o servicio que desarrollara en obra, es por esto que en la determinación de la proporción de la unidad cúbica de concreto, debe permitir un buen concreto con las facilidades manuales o mecánicas de que se disponga durante las etapas del proceso, transporte, colocación y compactación u otras propiedades que se consideran necesarias para el cual la mezcla esta diseñado.

Las propiedades del concreto fresco tiene variaciones, debido a una serie de factores tales como fuentes de abastecimiento de agregados, modificaciones en el tamaño nominal máximo, en la granulometría, diferentes tipos de cemento, cambios de volumen, variación de temperatura, método de mezclado, etc.

7.1.1 ENSAYO DE ASENTAMIENTO – NTP 339.035:1999

Este ensayo llamado también Slump, es un método que tiene aceptación universal y su merito principal reside en la sencillez de la operación, con una capacidad para detectar variaciones en la uniformidad de una mezcla de proporciones nominales especificas.

Entendemos por trabajabilidad del concreto a la mayor, o menor dificultad para el mezclado, transporte, colocación y compactación de la misma. El asentamiento es casi la primera prueba que se le hace al concreto, mostrando utilidad para evaluar la idoneidad de las mezclas en la consolidación en diferentes tipos de estructuras.

El comportamiento del concreto en la prueba indica su "consistencia" o sea, su capacidad para adaptarse al encofrado o molde con facilidad, manteniéndose homogéneo con un mínimo de vacíos.

El ensayo consiste en consolidar una muestra de concreto fresco en un molde troncocónico, midiendo el asiento del concreto después de desmoldarla.

PROCEDIMIENTO DEL ENSAYO

a) Se humedece el interior del molde y la base sobre la cual se va hacer el ensayo; la base debe ser firme, plana nivelada y no absorbente, recomendable plancha metálica.
b) Sujetar firmemente el molde con los pies, para ello pisar las aletas con que cuenta el molde.
c) Luego echar concreto al molde con el cucharón, hasta.1/3 del volumen del cono, altura aproximada de 6.5 cm, y chucear 25 veces de afuera hacia adentro en forma de espiral.
d) Enseguida colocar la segunda capa, altura aproximada de 15.5 cm. teniendo cuidado de que la varilla compactadora penetre ligeramente en la capa anterior.
e) Colocar la tercera capa, colocando un poco mas del concreto necesario y chuzar 25 veces, penetrando ligeramente en la capa anterior.
f) Eliminar el exceso de concreto usando una plancha y se aparta el concreto que se haya depositado al pie del molde.
g) Golpear suavemente con la varilla compactadora una de las generatrices del cono, con el fin de producir la caída del concreto.
h) Levantar el molde verticalmente en 5 a 10 segundos, sin impartirle movimiento lateral o de torsión.
i) Colocar el molde al lado del concreto ensayado y se mide la distancia entre la varilla colocada horizontalmente sobre el molde y la cara superior del concreto, esta distancia en centímetros es lo que se denomina asentamiento.

7.1.2 ENSAYO DE EXUDACION - NTP 339.077:1981

Propiedad por la cual una parte del agua de mezcla se separa de la masa y sube hacia la superficie del concreto.

Es un caso típico de sedimentación en que los sólidos se asientan dentro de la masa plástica.

Esta influenciada por la cantidad de finos en los agregados y la finura del cemento, por lo que cuanto más fina es la molienda de este y mayor sea el porcentaje de material menor que la malla N°100, la exudación será menor pues retiene el agua de mezcla.

La exudación se produce inevitablemente en el concreto, pues es una propiedad inherente a su estructura, luego lo importante es evaluarla y controlarla en cuanto a los efectos negativos que pudiera tener.

PROCEDIMIENTO DEL ENSAYO

a) Verter la mezcla de concreto en el balde metálico de ½ p^3 en tres capas compactando cada capa con 25 golpes; luego de llenar, proceder a nivelar y alisar la superficie del recipiente.
b) Quitar aproximadamente 1" de espesor de mezcla.
c) Pesar el recipiente con la mezcla, a fin de obtener el peso de la muestra por diferencia.
d) Enseguida a fin de facilitar la extracción del agua de exudación, colocar un taco de 5 cm de altura debajo de la base del molde metálico, con el objeto de inclinarla y el agua exudada se junte.
e) Con una pipeta o jeringa extraer el agua exudada a intervalos de 10 minutos durante los primeros 40 minutos, y luego a intervalos de 30 minutos de allí en adelante hasta que cese la exudación. Después de extraer el agua exudada se regresa el recipiente a su posición original.
f) Cuando se requiere solamente el volumen total de agua exudada el procedimiento de extracción periódica puede ser omitido y la extracción se hará en una sola operación.

CALCULOS

Datos previos:

Peso de la muestra en Kg (Wmuestra)

Cantidad total de agua exudada (Wagua exudada)

De los datos de diseño:

Cantidad de agua por m^3 en Kg (Wagua diseño)

Peso total de materiales x m^3 de concreto (Wmateriales)

Con los datos antes mencionado se tiene:

$$\text{Wagua de la muestra} = \frac{\text{W muestra}}{\text{W materiales}} \times \text{Wagua diseño}$$

Luego, el Porcentaje de Exudación:

$$\text{\% de Exudacion} = \frac{\text{Wagua exudada}}{\text{Wagua de la muestra}} \times 100$$

Se calcula asimismo la velocidad de exudación en mm/hr, como el volumen del agua de exudación por intervalo de tiempo.

$$\text{Velocidad de exudacion} = \frac{\text{Volumen exudado}}{\text{Tiempo}}$$

Donde:

Volumen exudado = Volumen de agua de exudación por unidad de superficie

= Vi / A = Asentamiento acumulado

Vi = Volumen en cm^3 del agua exudada acumulada, durante el intervalo seleccionado.

A = Área expuesta del concreto en cm^2

7.1.3 ENSAYO DE PESO UNITARIO - NTP 339.046:1979

El Ensayo de Peso Unitario consiste en la determinación del peso de concreto por unidad de volumen; los determinantes en el valor del peso unitario son los pesos específicos de los agregados, pudiéndose clasificar en concretos Densos, Normales y Ligeros.

Según el tamaño máximo nominal del agregado grueso, el recipiente será de 14 dm^3 (½ pie^3) para agregados de hasta 2" y de 28 dm^3 (1 pie^3) para agregados de tamaño máximo mayores.

Para la presente tesis de investigación el tamaño nominal máximo obtenido es de 1", por lo que usaremos el recipiente de ½ pie^3.

PROCEDIMIENTO DEL ENSAYO

a) Llenar con concreto el recipiente hasta un tercio de su capacidad para luego compactarla con 25 golpes; de la misma manera se llenan las 02 capas restantes y se realiza el mismo paso descrito anteriormente.
b) En la última capa se coloca material en exceso para enrasar a tope con el material, para luego golpear ligeramente la superficie exterior del recipiente con el fin de "eliminar" los vacíos que pudieran haber quedado.
c) Se procede a pesar el recipiente.

CALCULO

El peso unitario se calcula dividiendo el peso neto del concreto entre el volumen del recipiente.

$$P_{\text{unitario}} = P_C \,/\, V_R$$

Donde:

P_R = Peso del concreto neto

V_B = Volumen del recipiente.

7.1.4 ENSAYO DE TIEMPO DE FRAGUADO - NTP 339.082:2001

El fraguado es el proceso de endurecimiento del concreto. Por lo que su determinación tiene una trascendencia muy importante, por cuanto nos da la pauta del tiempo que se dispone en el proceso constructivo para las operaciones de colocación y acabado.

Este proceso esta dividido en dos periodos:

El fraguado inicial
El fraguado final

El fraguado inicial se caracteriza por la perdida de plasticidad y aumento en la temperatura de la mezcla.

El fraguado final se caracteriza por endurecimiento significativo y deformaciones permanentes, como lógica consecuencia del aumento de su resistencia.

La fragua del concreto depende básicamente del contenido de aluminato tricalcico del cemento, finura del cemento, relación agua/cemento, temperatura y humedad del ensayo.

La norma establece el tiempo de fraguado del concreto con asentamiento superior a cero por medio de agujas de penetración sobre la muestra tamizada; así el fraguado inicial se determina por el tiempo transcurrido, luego del contacto inicial del cemento y el agua hasta que el mortero alcance una resistencia a la penetración de 500 $lb/pulg^2$, y la fragua final cuando alcance 4000 $lb/pulg^2$.

La resistencia a la penetración, se calcula como el cociente de la fuerza requerida para que la aguja penetre 25mm y el área de la superficie de contacto de la aguja.

PROCEDIMIENTO DEL ENSAYO

a) Se usara dos moldes cilíndricos para el ensayo, ellos deben tener 15 cm de diámetro y de 15 cm de altura.
b) Debe contarse con un aparato hidráulico con capacidad de 60 kgf a 100 kgf provisto de un dispositivo medidor de presión y un medidor de carga con escala graduada.
c) Preparamos una tanda de 0.02 m^3 de concreto.
d) Tamizamos la mezcla por la malla N° 4; la mezcla que pasa por dicha malla, mortero, es llenada en los dos moldes cilíndricos.
e) Se llena cada molde en dos capas con 27 golpes cada capa hasta una altura mínima de 14 cm. Se golpea a los costados del molde para eliminar las burbujas de aire y luego se enrasa.
f) Se dispone de 6 agujas cuyos diámetros son de 1 1/8", 13/16", 9/16", 5/16", 4/16", y 3/16".
g) La muestra debe almacenarse a temperatura ambiente y protegerse del sol para evitar el secado inmediato. Antes del ensayo con la ayuda de una pipeta retirar el agua que haya exudado.
h) Se anota la hora de inicio del ensayo.
i) Según el estado de endurecimiento del mortero, se debe colocar el aparato con una aguja de tamaño apropiado y se pone esta en contacto con el mortero.
j) Se aplica una fuerza vertical y uniformemente hacia abajo hasta lograra una penetración de 25 mm. En un tiempo aproximado de 10 segundos.
k) Se registra la fuerza aplicada, el área de la aguja de penetración y la hora de ensayo. En posteriores ensayos de penetración se debe tener cuidado en eludir sitios en los cuales el mortero ha sido alterado por penetraciones previas.
l) La distancia libre entre la aguja y el lugar de cualquier penetración anterior, debe ser al menos 2 veces el diámetro de la aguja que se use, pero en ningún caso inferior a 15 mm. Se debe dejar una distancia libre entre la aguja y la pared del recipiente por lo menos de 25 mm.
m) Para muestras normales y temperaturas normales, el primer ensayo se debe hacer cuando haya transcurrido 3 a 4 horas y los demás ensayos cada hora. Para mezclas aceleradas o altas temperaturas se recomienda hacer el primer ensayo

cuando hayan transcurrido 1 a 2 horas y los demás ensayos a intervalos de 0.5 horas.

n) Para condiciones de baja temperatura o mezcla retardantes, el primer ensayo hacer cuando hayan transcurrido 4 horas o más, los posteriores deben llevarse a intervalos de 1 hora a menos que el incremento de resistencia a la penetración indique que son aconsejables a intervalos mas cortos.

o) Para cada ensayo de fraguado, se deben hacer por lo menos 6 penetraciones y los intervalos de tiempo entre ellas, serán tales que suministren pintos adecuados y lo suficientemente espaciados para dibujar una curva satisfactoria de velocidad de endurecimiento.

CALCULO

Se calcula la resistencia a la penetración en lb / pulg^2, como cociente de la fuerza requerida y el área de la aguja utilizada ver cuadros de calculo del tiempo de fraguado.

$$P = F / A$$

Donde :

P	:	Resistencia a la penetración en lb/ pulg^2
F	:	Fuerza necesaria para penetrar 25mm
A	:	Area de contacto de la aguja con el mortero.

7.1.5 ENSAYO DE FLUIDEZ - NTP 339.085:1981

El ensayo de fluidez mide la resistencia que opone el concreto al experimentar deformaciones, la cual depende de la forma, gradación, tamaño máximo del agregado en la mezcla y de la cantidad de agua en la mezcla.

En la presente tesis el método de ensayo usado fue de la mesa de sacudida (NORMA ITINTEC 339.085). Este método se considera aplicable a concretos plásticos que tienen agregados grueso hasta 38mm (1 ½"). Si el agregado grueso es mayor de 38 mm (1 ½"); el método es aplicable cuando se realiza sobre la porción de hormigón que pasa el tamiz ITINTEC 38 mm (1 ½") después de haber eliminado los agregados mayores de acuerdo como se indica en la Norma ASTM C-172.

Como sabemos la consistencia es el grado de fluidez de una mezcla; determinada de acuerdo a un procedimiento prefijado.

PROCEDIMIENTO DEL ENSAYO

a) El ensayo consiste en determinar el aumento del diámetro que experimenta la base inferior de un tronco de cono de masa de concreto fresco, sometido a sacudidas sucesivas.
b) Previamente limpiar la mesa de sacudidas, quitando el exceso de agua con una esponja.
c) Enseguida centrar el molde sobre la mesa, se sujeta firmemente y luego echar material suficiente hasta la mitad del molde.
d) Con ayuda de la varilla compactadora aplicar veinticinco golpes, distribuidos uniformemente, por toda la sección de la masa.
e) Luego llenar el molde con exceso y aplicar otros veinticinco golpes con la varilla, procurando que esta penetre hasta la capa inferior y que la masa rellene todos los huecos.
f) Retirar el concreto sobrante y limpiar la mesa.
g) Sacar el molde con cuidado, levantarlo verticalmente y lo más rápido posible.
h) Luego se eleva y se deja caer durante 15 veces, desde una altura de 12.5 mm en 15 segundos girando la manivela con una velocidad uniforme.
i) Se determina el índice de consistencia calculando el tanto por ciento del aumento del diámetro, expresado en centímetros, de la base inferior del tronco de cono.
j) Se toma como diámetro medio del concreto extendido, la media aritmética de seis mediciones del diámetro, distribuidas simétricamente.

CALCULO

$$F = \frac{(D - 25)}{25} \times 100$$

Donde

D	:	Diámetro promedio
F	:	Factor de asentamiento.

7.1.6 ENSAYO DE CONTENIDO DE AIRE - NTP 339.080:1981

El contenido de aire en el concreto es el porcentaje de vacíos que hay en la misma.

En todo concreto se encuentra aire y es por eso que es importante cuantificarla para prevenir los efectos perjudiciales de esta, tal como la reducción de la resistencia del concreto por incremento de la porosidad del mismo.

El contenido de aire dependerá del tamaño máximo nominal del agregado, es decir, a medida que aumenta ese tamaño, se incrementara el contenido de aire.

PROCEDIMIENTO DEL ENSAYO

El procedimiento para encontrar el contenido de aire es mediante el método de presión "Aparato Washington". El aparato consta de dos partes, en una se encuentra la cámara donde se almacena el aire a presión y el manómetro que indica la cantidad de agua penetrada en el concreto o presión de aire, lo cual se puede deducir que será el mismo porcentaje de vacíos que tendrá el concreto, la otra parte del aparato es un molde cilíndrico donde se coloca el concreto, similar al peso unitario.

a) Se coloca el concreto el concreto en el recipiente en tres capas con 25 golpes por cada capa, se enrasa la superficie de modo que no quede burbujas de aire en la superficie.
b) Se coloca la tapa y se procede a llenar de aire la cámara de presión
c) Luego por unas aberturas se introduce agua al concreto y se procede luego a cerrar las aberturas.
d) Se abre la llave que une la cámara de aire con el concreto y la presión del aire hace introducir el agua en los vacíos de concreto lo que el manómetro indica

Los resultados de los ensayos del concreto al estado fresco, se muestran en el capítulo 8. El detalle de calculo se encuentra en el ANEXO C.

7.2 ENSAYOS EN EL CONCRETO ENDURECIDO

La estructura interna del concreto endurecido consistente en el aglomerante, estructura básica o matriz, constituida por la pasta de cemento y agua, que aglutina a los agregados gruesos, finos, aire y vacíos, estableciendo un comportamiento resistente debido en gran parte a la capacidad de la pasta para adherirse a los agregados y soportar esfuerzos de tracción y compresión, así como a un efecto puramente mecánico propiciado por el acomodo de las partículas inertes y sus características propias.

Por lo tanto la estructura del concreto no es homogénea, y en consecuencia no es isotrópica, es decir no mantiene las mismas propiedades en diferentes direcciones.

Un aspecto sumamente importante en la estructura del concreto endurecido reside en la porosidad o sistema de vacíos. Gran parte del agua interviene en la mezcla, solo cumple la función de lubricante en el estado plástico, ubicándose en líneas de flujo y zonas de sedimentación de los sólidos, de manera que al producirse el endurecimiento y evaporarse, quedan los vacíos o poros, que condicionan el comportamiento posterior del concreto para absorber líquidos y su permeabilidad o capacidad de flujo a través de él.

Los ensayos que permiten determinar las propiedades del concreto al estado endurecido, y por ende controlar la calidad del concreto, deben efectuarse de acuerdo a las normas, debido a que resultados erróneos pueden llevar al cuestionamiento de la calidad del concreto.

7.2.1 ENSAYO DE RESISTENCIA A LA COMPRESIÓN NTP 339.034:1999

Es la capacidad de soportar cargas y esfuerzos, siendo su mejor comportamiento en compresión en comparación con la tracción, debido a las propiedades adherentes de la pasta de cemento.

Un factor indirecto pero no por eso menos importante en la resistencia, lo constituye el curado ya que es el complemento del proceso de hidratación sin el cual no se llegan a desarrollar completamente las características resistentes del concreto.

PROCEDIMIENTO DE ENSAYO

a) Una vez elaborada la mezcla de concreto, se procede a llenar las probetas de 15x30 cm en tres capas, compactando cada capa con 25 golpes verticales mediante una varilla lisa de 5/8" con punta semiesférica, uniformemente repartidos de afuera hacia adentro en forma de espiral.
b) Después de llenar el molde, se procede a golpear suavemente las paredes del molde, utilizando la varilla, para eliminar los vacios que pudieran haber quedado.
c) Se enrasa la superficie del molde, a fin de obtener una superficie plana.
d) Las probetas deberán retirarse del molde al cabo de 20 h ± 4 h, después de elaborados. En estas horas iniciales, se deben almacenar sobre una superficie horizontal, evitando golpes o vibraciones.
e) Después de retiradas del molde las probetas deben almacenarse a temperatura permanente entre 23°C ± 2°C y bajo condiciones de humedad tales que siempre se mantenga agua libre en toda su superficie (por ejemplo sumergidos totalmente en agua saturada de cal).
f) Para conseguir la aplicación uniforme de la carga por parte de la prensa hidráulica, se procede a refrendar los extremos de las probetas empleando una mezcla de azufre y de material granuloso (capping).

La norma del ACI especifica para una prueba de resistencia el promedio de dos cilindros de la misma muestra probada a la misma edad, el cual normalmente es de 28 días.

7.2.2 ENSAYO DE RESISTENCIA A LA TRACCION POR COMPRESION DIAMETRAL NTP 339.084:1981

La resistencia a la tracción del concreto es relativamente baja. Una buena aproximación para la resistencia a la tracción f_{ct} es $0.10f'c < f_{ct} < 0.20f'c$. Es más difícil medir la resistencia a la tracción que la resistencia a compresión debido a los problemas de agarre con las maquinas de prueba. Debido a la existencia de diversos métodos que requieren una operación compleja, se optó por el método de tracción por hendimiento o prueba brasileña consistente en romper un cilindro de concreto, del tipo normalizado para el ensayo de compresión, entre los cabezales de una prensa, según generatrices opuestas.

La resistencia a la tracción debe darse según la relación:

$$T = \frac{2 \times P}{\Pi \times D \times L}$$

Donde:

P	=	Fuerza de compresión
D	=	Diámetro
L	=	Longitud del cilindro.

PROCEDIMIENTO DE ENSAYO

a) Antes de la prueba debe procederse a determinar la longitud.
b) Si las dimensiones de las placas de apoyo de la maquina de compresión, son menores que la longitud del cilindro, debe interponerse una platina suplementaria de acero maquinado, de por lo menos 50 mm de ancho y espesor no menor que la distancia entre el borde de las placas de l apoyo y el extremo del cilindro.
c) Debe colocarse entre el cilindro y la superficie de los cabezales de la maquina de ensayo, o eventualmente la platina suplementaria de ser utilizada, tablillas de madera contraplacadas, de 3 mm de espesor y 25mm de ancho. A lo largo de toda la longitud del cilindro, con el fin de que la probeta al momento de realizar la prueba se mantenga quieta.
d) Se aplica la carga a la probeta con una velocidad en forma continua, evitando el impacto.
e) La velocidad de aplicación de la carga indicada para probetas normales esta comprendida entre 5000 y 1000 da N/min hasta la rotura.

7.2.3 ENSAYO DE MÓDULO DE ELASTICIDAD ESTÁTICO ASTM 496-63

En general es la capacidad del concreto de deformarse bajo carga, sin tener deformación permanente.

El concreto no es un material elástico estrictamente hablando, ya que no tiene comportamiento lineal en ningún tramo de su diagrama carga vs deformación en compresión, sin embargo, convencionalmente se acostumbra definir un "Módulo de Elasticidad Estático" del concreto mediante una recta tangente a la parte inicial del diagrama, o a una recta secante que une el origen del diagrama con un punto establecido que normalmente es un % de la tensión ultima.

Los módulos de elasticidad normales oscilan entre 250,000 a 350,000 kg/cm^2 y están en relación directa con la resistencia a la compresión del concreto y por ende con la relación agua/cemento. Conceptualmente, las mezclas más ricas tienen módulo de elasticidad mayores y mayor capacidad de deformación de las mezclas pobres.

Al someterse una probeta de concreto a una carga que se incrementa constantemente, ocurre una deformación plástica o escurrimiento.

La curva esfuerzo - deformación muestra una zona de trabajo donde los esfuerzos y las deformaciones son proporcionales para fines prácticos.

Este limite de proporcionalidad para el caso del módulo de elasticidad es el 40% de la resistencia a la compresión y la deformación para este punto.

Es importante decir que la deformación del módulo elástico es una aproximación por cualquiera de los métodos que existen; sencillamente por que el concreto no es perfectamente elástico.

Como el concreto no es un material linealmente elástico, en ningún momento sigue la ley de Hooke, es decir que el diagrama esfuerzo deformación no presenta ningún tramo recto. De manera que el "pseudo Módulo de Elasticidad", es la pendiente de la secante a la curva carga vs deformación desde el origen a un punto de tensión determinada (generalmente la tensión de trabajo).

Para esfuerzos de trabajo pequeños y alternantes el módulo en el origen puede tomarse como el módulo de elasticidad dinámico.

El módulo de elasticidad del concreto Ec es una función compleja de muchas variables como la tensión de trabajo, forma de solicitación, duración de las cargas, estado higroscópico, etc.

El ACI sugiere la siguiente expresión para su calculo:

$$Ec = W^{1.5}*4270*(f'c)^{0.5}$$

Donde:

W = Peso especifico del concreto t/m^3.

f'c = Resistencia en kg/cm^2.

Existen varios métodos como el mencionado anteriormente; para la presente tesis de investigación se ha utilizado el equipo de los Espejos de Martens.

PROCEDIMIENTO DE ENSAYO

a) Se instala el equipo de los espejos de Martens convenientemente centrado con respecto al eje de la probeta y teniendo especial cuidado en colocar el eje de los espejos en las dos generatrices opuestas que pasan por el plano central de la probeta, al final la distancia entre los espejos y la regla de lectura debe de ser de 1.25 cm.
b) Aplicar la carga continuamente y sin sacudidas; para cada espécimen se tomara lecturas cada 2000 kg, hasta llegar a la carga máxima de rotura. Se anotaran dos lecturas, una del lente derecho y otra del lente izquierdo.
c) Se trabaja con el promedio de las dos lecturas corregidas (menos su referencia). Se confeccionará un gráfico, Esfuerzo vs Deformación Unitaria Longitudinal.

Se calcula el Módulo de Elasticidad como sigue:

$$E = \frac{(E2 - E1)}{(D2-0.5x10^{-4})}$$

Donde:

E = Módulo de elasticidad estático (10^5 kg/cm^2)

E2 = Esfuerzo correspondiente al 40% de la carga ultima

E1 = Esfuerzo correspondiente a una deformación longitudinal D1, de 0.5×10^{-4} (kg/cm^2)

D2 = Deformación longitudinal producida por el esfuerzo E2.

7.2.4 ENSAYO DE RESISTENCIA A LA FLEXIÓN EN VIGAS NTP 339.078:2001

Como es difícil aplicar tensión uniaxial al espécimen de concreto, la resistencia a la tracción del concreto se determina por métodos indirectos: la prueba de flexión y la prueba de cuarteadura, para este tema de investigación se utilizara el primero, estos métodos producen valores de resistencia que son mayores que la resistencia a la tracción real bajo carga uniaxial.

En la prueba de flexión, el esfuerzo a la tensión máxima teórica alcanzada en la fibra del fondo de una viga de prueba se conoce como modulo de ruptura. El valor del modulo de ruptura depende las de la viga y sobre todo de la disposición de la carga. Hoy en día la carga simétrica en dos puntos (a los tercios de la luz) se usa en Estados unidos de América. Esto produce un momento de flexión constante entre los puntos de carga, de modo que un tercio de la luz esta sujeto al esfuerzo máximo y por tanto, es ahí donde probablemente se produzca el agrietamiento.

PROCEDIMIENTO DE ENSAYO

a) Las dimensiones de los especimenes rectangulares, utilizados para el ensayo de flexión son de 15 x 15 x 75 cms.
b) Se coloca con respecto a la posición de vaciado y se centra con respecto a las placas de apoyo. Las placas de aplicación de carga se ponen en contacto con la muestra y sobre los punto extremos del tercio central de la luz libre.
c) Las dimensiones serán tomadas con una aproximación de 1mm con la finalidad de determinar el promedio en la sección de falla.
d) Si la fractura ocurre dentro del tercio medio central de la viga, se calcula el modulo de ruptura, con base en toeoria elastica ordinaria como sigue:

$$Mr = \frac{PL}{bh^2}$$

Donde:

Mr = Modulo de ruptura en kg/cm^2

P = Carga máxima en kg

L = Luz en cms

b = Ancho promedio del espécimen rectangular en cms

h = Altura promedio del espécimen rectangular en cms.

e) Si la fractura se produce fuera del tercion medio central entonces el resultado de la prueba de be descartarse. Por otro lado, ASTM C78-84 permite la falla fuera de los puntos de carga a una distancia promedio ***a*** del apoyo mas cercano, por medio de la ecuación:

$$Mr = \frac{3Pa}{bh^2}$$

Donde:

a = Distancia entre la línea de falla y el apoyo mas cercano.

f) Sin embargo si la falla ocurre en una sección, tal que (L/3 – a) > 0.05L, entonces se debe descartar el ensayo.

7.2.5 ENSAYO DE RESISTENCIA AL IMPACTO ACI-542

La resistencia al impacto del concreto reforzado con fibra de acero, es medido por una prueba el cual se usa un martillo de 4.5 kg que cae sobre una bola de acero centrado sobre una muestra de 1.5 a 2.5 plg de grosor por 6 plg de diámetro. El numero de golpes requerido para romper y separar una muestra de fibra a una edad de 28 días es de 200 a 500 o mas golpes dependiendo de la configuración extensión y cantidad de la fibra. La muestra de shotcrete simple normalmente sede de 10 a 40 golpes.

PROCEDIMIENTO DE ENSAYO

a) Se construyeron moldes cilíndricos (discos) de 6 plg. de diámetro interior y con una altura de 2 ½ plg.. El procedimiento fue vaciar una sola capa y compactarla mediante 25 golpes con una varilla de 5/8 plg. de diámetro, uniformemente distribuidas en la sección del recipiente y enrasarlos.

b) Al día siguiente de vaciado los moldes se procedió a desmoldar y pasar los discos cilíndricos la poza de curado hasta el día de ensayo.

c) Una vez que los discos cilíndricos estén fuera de la poza de curado se dejo secar a temperatura ambiente para luego proceder a ensayarlos mediante una carga de Impacto, La carga fue entregada por un peso de 10 libras (455 grs) desde una altura de 18 plg. (45.72 cm).

d) Se procedió a contar el numero de golpes al producirse la primera grieta así como el instante de la falla las cuales fueron registradas.

CAPÍTULO 8

RESULTADOS DE LOS ENSAYOS

8.1 ENSAYOS EN EL CONCRETO PATRÓN

8.1.1 ENSAYO EN EL CONCRETO FRESCO

8.1.1.1 ENSAYO DE ASENTAMIENTO (plg)

Relación a/c	Asentamiento
0.60	5"
0.65	5"
0.70	5"

8.1.1.2 ENSAYO DE EXUDACIÓN (%)

Relación a/c	Muestra	%Exudación	Promedio
0.60	M-1	2.66	2.58
	M-2	2.51	
0.65	M-1	2.77	2.77
	M-2	2.78	
0.70	M-1	2.71	2.83
	M-2	2.94	

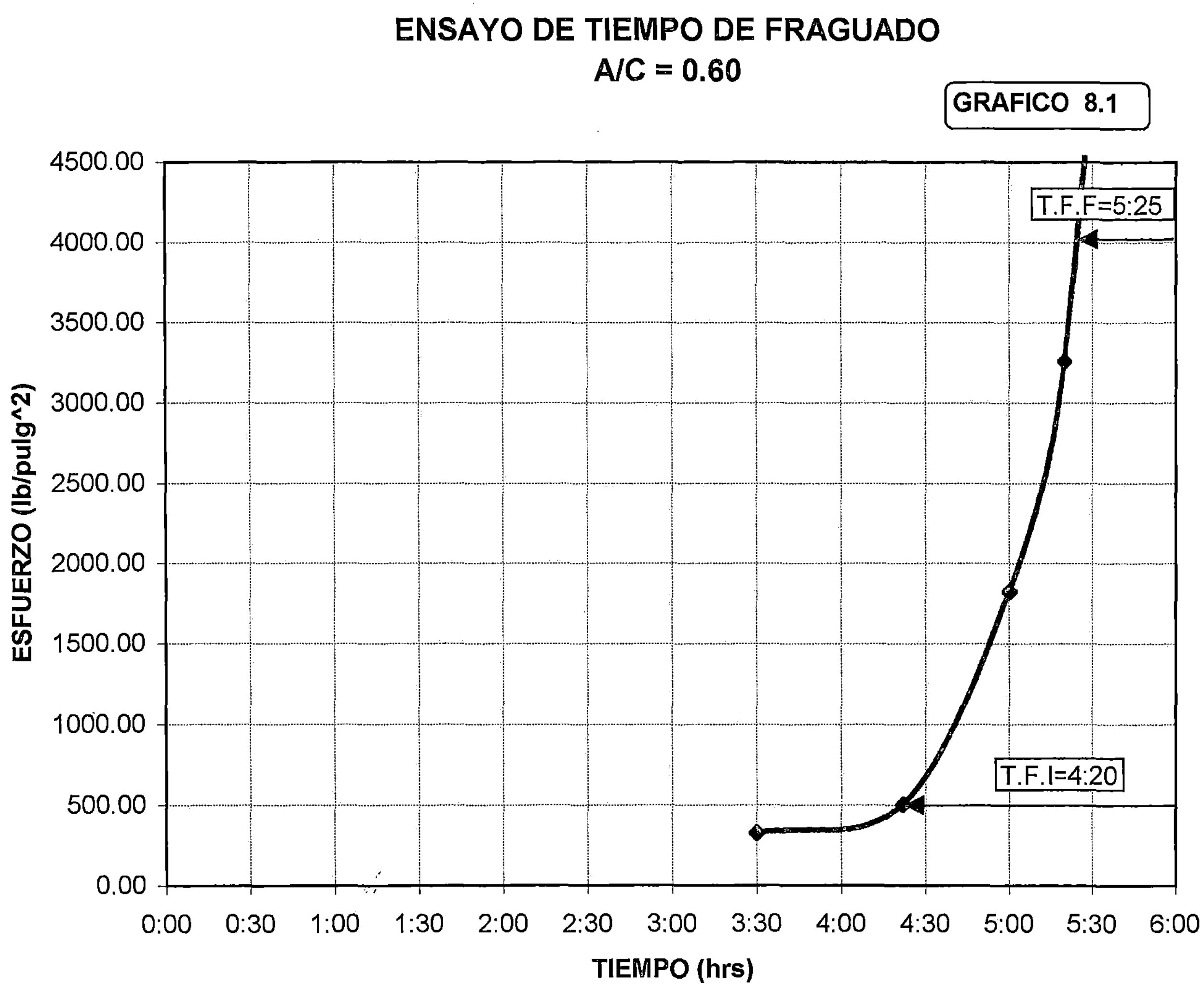
8.1.1.3 ENSAYO DE PESO UNITARIO (kg/m^3)

Relación a/c	Peso Unitario Compactado (Kg/m^3)
0.60	2442.86
0.65	2450.00
0.70	2450.00

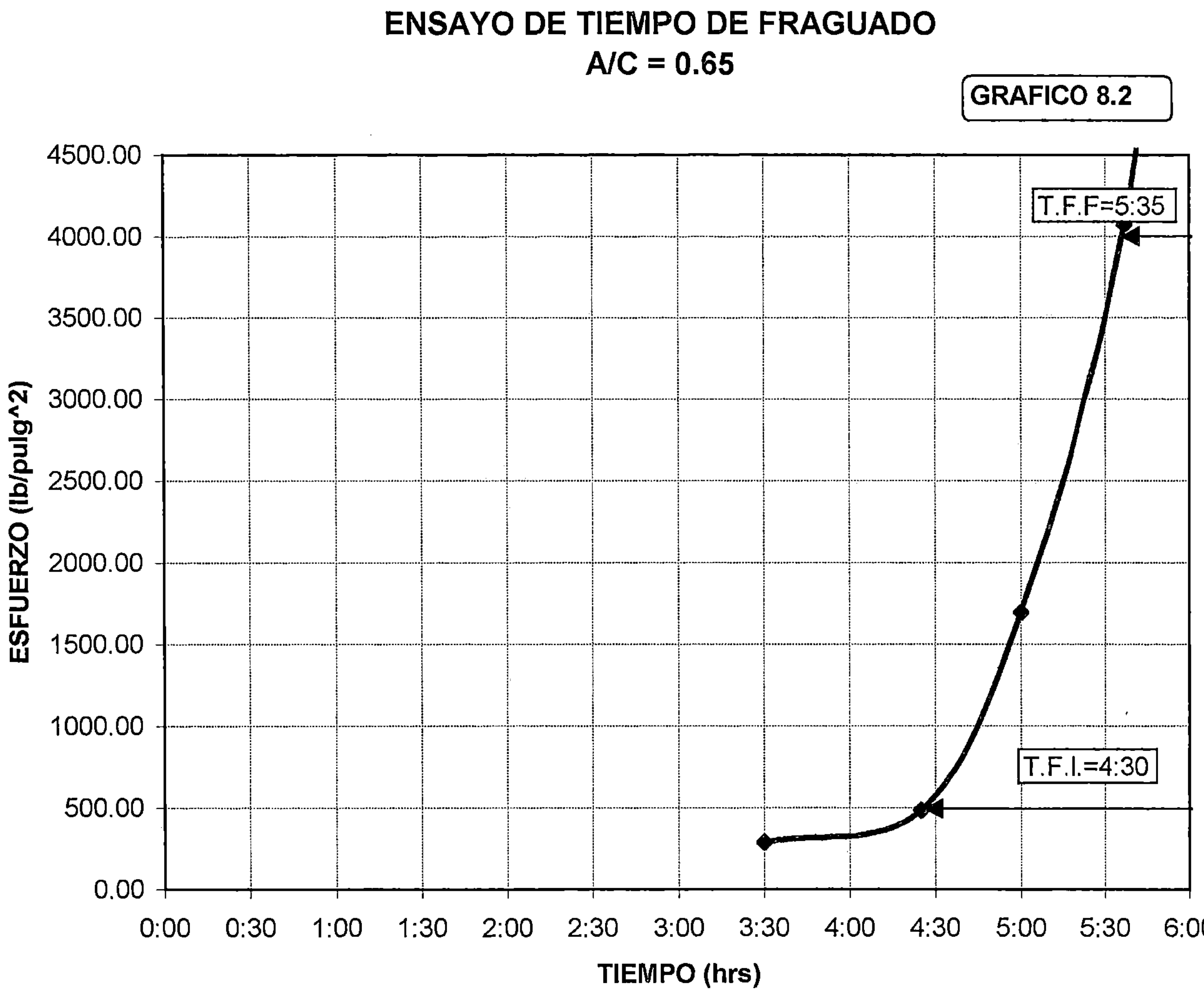
8.1.1.4 ENSAYO DE TIEMPO DE FRAGUADO (Hr:min)

Relación a/c	Tiempo (Hr)	Fuerza (Lbs)	Sección (Plg^2)	Resistencia (Lbs/plg^2)	Fragua Inicial	Fragua Final
0.60	0:00			0.00	4:20	5:25
	03:00	130	0.99402	130.78		
	03:30	170	0.51848	327.88		
	04:22	125	0.24850	503.02		
	05:00	140	0.07669	1825.53		
	05:30	160	0.04908	3259.98		
	05:40	200	0.02761	7243.75		
0.65	00:00			0.00	4:30	5:35
	03:00	150	0.99402	150.90		
	03:30	150	0.51848	289.31		
	04:25	120	0.24850	482.90		
	05:00	130	0.07669	1695.14		
	05:37	200	0.04908	4074.98		
	06:00	200	0.02761	7243.75		
0.70	00:00			0.00	4:35	5:45
	03:00	130	0.99402	130.78		
	04:00	155	0.51848	298.95		
	04:37	125	0.24850	503.02		
	05:00	150	0.07669	1955.93		
	05:48	200	0.04908	4074.98		
	06:20	195	0.02761	7062.66		

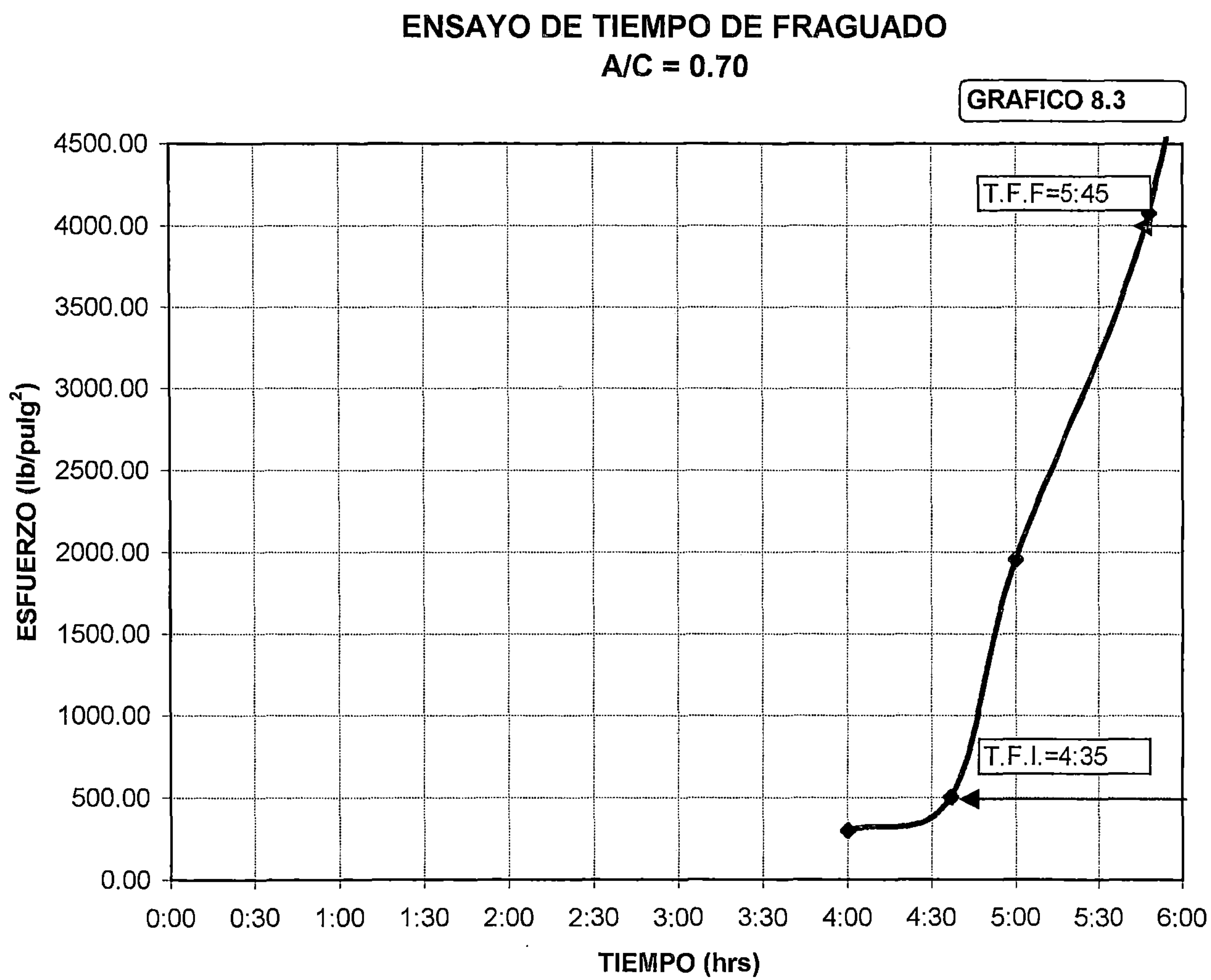
CONCRETO PATRÓN



CONCRETO PATRÓN



CONCRETO PATRÓN



8.1.1.5 ENSAYO DE FLUIDEZ (%)

Relación a/c	Fluidez (%)
0.60	105.71
0.65	110.83
0.70	113.91

8.1.1.6 ENSAYO DE CONTENIDO DE AIRE (%)

Relación a/c	Contenido de Aire (%)
0.60	1.60
0.65	1.50
0.70	1.40

8.1.2 ENSAYOS EN EL CONCRETO ENDURECIDO

8.1.2.1 ENSAYO DE RESISTENCIA A LA COMPRESIÓN (kg/cm^2)

RELACIÓN AGUA CEMENTO 0.60

N° Dias	Carga (Kg)	Diametro (cm)	Resistencia (Kg/cm^2)	Promedio (Kg/cm^2)
7	33800	15	191.27	183.01
7	31400	14.9	180.08	
7	31400	15.00	177.69	
14	39100	14.90	224.24	230.82
14	39200	14.86	226.03	
14	42800	15.00	242.20	
28	56800	15.20	313.02	311.97
28	55000	15.10	307.13	
28	55800	15.00	315.76	
28	55400	15.20	305.30	
28	57100	15.00	323.12	
28	55800	15.20	307.51	
42	60600	15.20	333.96	333.77
42	60000	15.10	335.05	
42	60300	15.20	332.31	

RELACIÓN AGUA CEMENTO 0.65

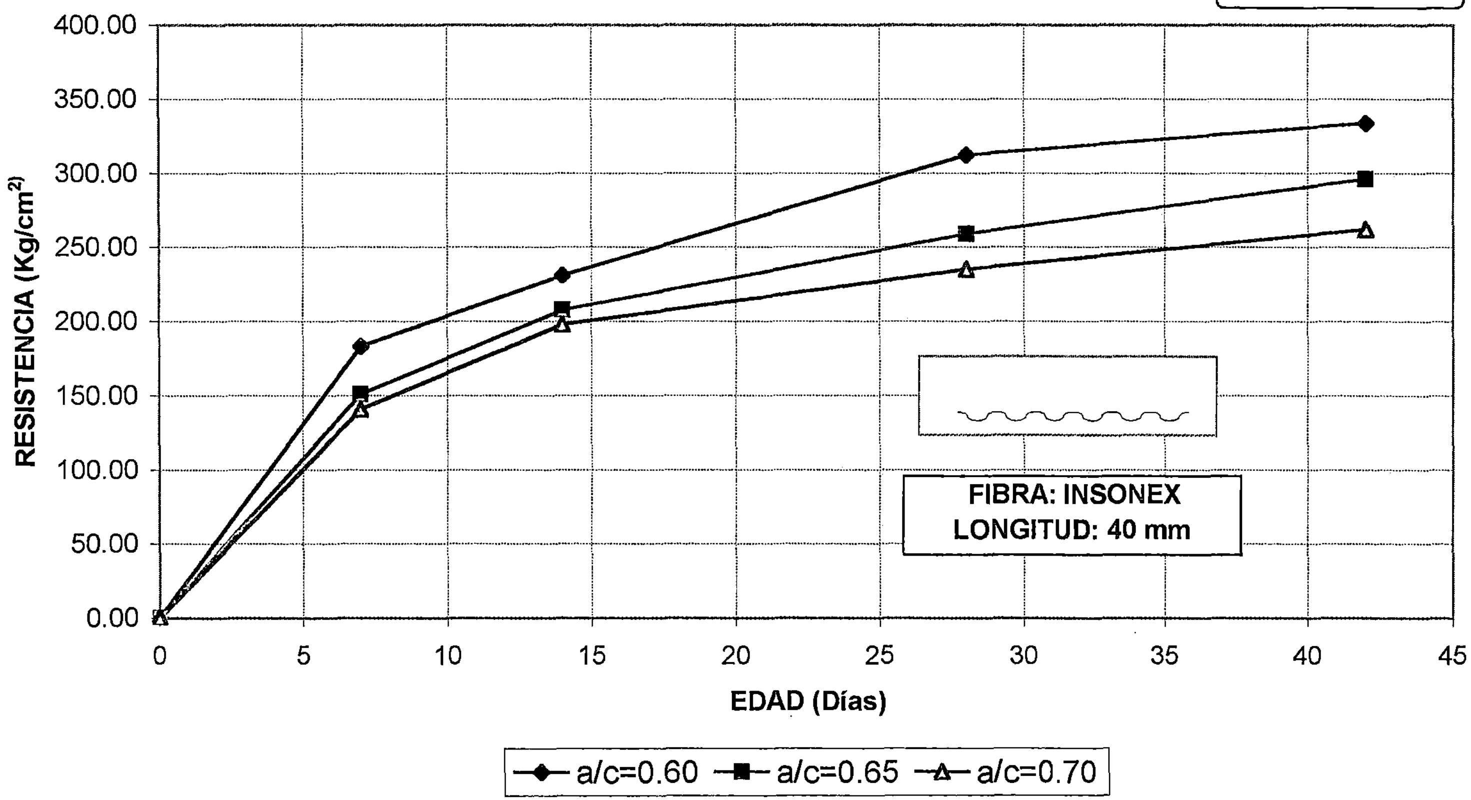
N° Dias	Carga (Kg)	Diametro (cm)	Resistencia (Kg/cm^2)	Promedio (Kg/cm^2)
7	25800	15.00	146.00	150.62
7	28600	15.20	157.61	
7	26200	15.00	148.26	
14	33100	14.80	192.40	207.49
14	35500	15.00	200.89	
14	39800	14.87	229.18	
28	48400	15.20	266.73	258.65
28	45900	15.20	252.95	
28	45800	15.00	259.17	
28	44800	14.94	255.56	
28	45000	15.00	254.65	
28	47700	15.20	262.87	
42	51600	14.80	299.94	296.00
42	51600	15.00	292.00	
42	51900	14.94	296.06	

RELACIÓN AGUA CEMENTO 0.70

N° Dias	Carga (Kg)	Diametro (cm)	Resistencia (Kg/cm^2)	Promedio (Kg/cm^2)
7	24800	14.87	142.80	140.41
7	25200	14.97	143.17	
7	23900	15.00	135.25	
14	33900	14.95	193.12	198.01
14	34100	14.82	197.68	
14	35200	14.85	203.24	
28	39900	15.00	225.79	234.90
28	41300	15.10	230.62	
28	39600	15.00	224.09	
28	44500	14.95	253.51	
28	39000	14.94	222.47	
28	44100	14.90	252.92	
42	49200	14.80	285.99	261.85
42	41400	14.95	235.85	
42	46600	15.00	263.70	

RESISTENCIA A LA COMPRESIÓN PARA LAS DIFERENTES PROPORCIONES a/c CONCRETO PATRÓN

GRAFICO 8.4



Tesis: "Estudio del comportamiento del concreto de mediana a baja resistencia, con la incorporación de Fibras de Acero y CPT I Andino"

8.1.2.2 ENSAYO DE RESISTENCIA A LA TRACCIÓN POR COMPRESIÓN DIAMETRAL (kg/cm^2)

RELACIÓN AGUA CEMENTO 0.60

N° Dias	Carga (Kg)	Diametro (cm)	Luz (cm)	Resistencia (Kg/cm^2)	Promedio (Kg/cm^2)
28	21000	14.93	30.10	29.76	29.58
28	20800	15.02	30.00	29.39	

RELACIÓN AGUA CEMENTO 0.65

N° Dias	Carga (Kg)	Diametro (cm)	Luz (cm)	Resistencia (Kg/cm^2)	Promedio (Kg/cm^2)
28	16400	14.82	30.16	23.36	25.04
28	18900	14.98	30.05	26.73	

RELACIÓN AGUA CEMENTO 0.70

N° Dias	Carga (Kg)	Diametro (cm)	Luz (cm)	Resistencia (Kg/cm^2)	Promedio (Kg/cm^2)
28	16100	14.95	30.16	22.73	22.73
28	15900	14.83	30.03	22.73	

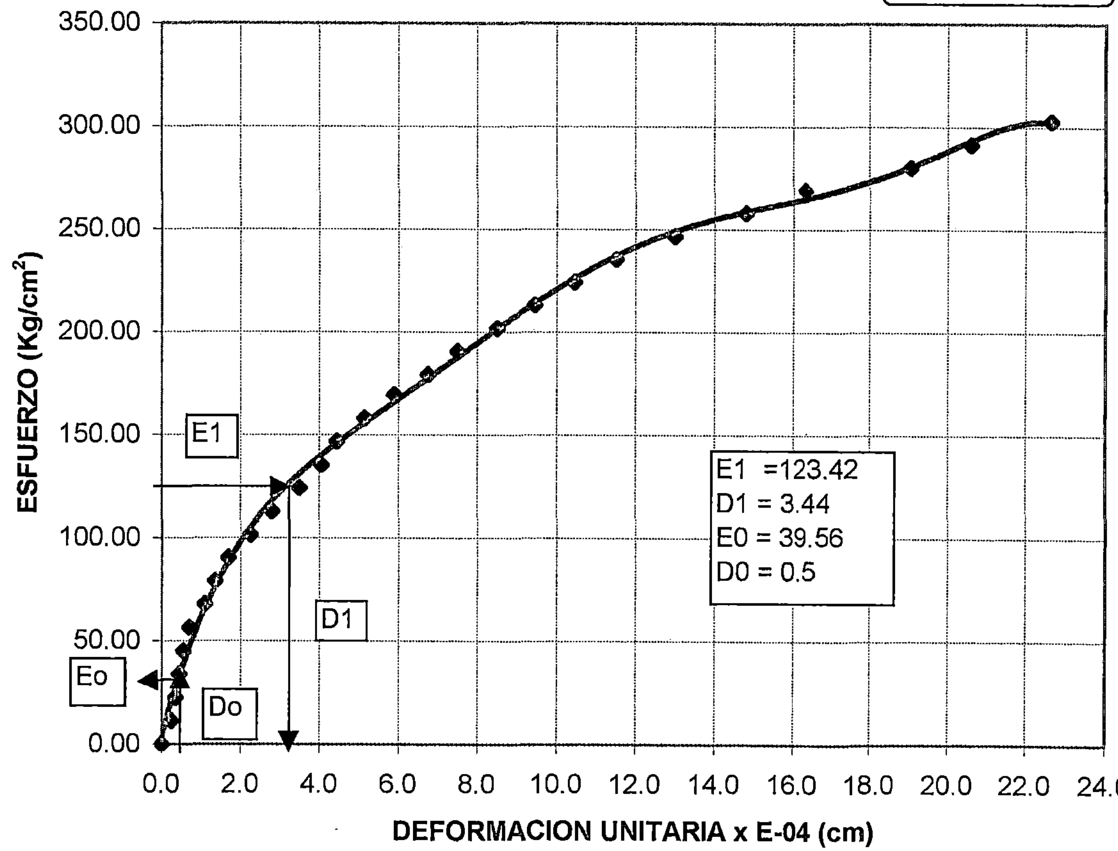
8.1.2.3 ENSAYO DE MÓDULO DE ELASTICIDAD ESTÁTICO (10^5 kg/cm^2)

Relación a/c	Modulo Elastico Estatico x100000 kg/cm^2
0.60	2.8530
0.65	2.7010
0.70	2.5660

Esfuerzo	Def.Unit
0.00	0.00
11.30	0.25
22.61	0.35
33.91	0.45
45.21	0.55
56.51	0.70
67.82	1.10
79.12	1.35
90.42	1.70
101.72	2.25
113.03	2.80
124.33	3.50
135.63	4.05
146.93	4.45
158.24	5.15
169.54	5.90
179.37	6.75
190.58	7.50
201.79	8.50
213.00	9.45
224.22	10.45
235.43	11.50
246.64	13.00
257.85	14.80
269.06	16.35
280.27	19.05
291.48	20.60
302.69	22.65

MÓDULO ELASTICO ESTÁTICO CONCRETO PATRÓN
RELACIÓN AGUA CEMENTO 0.60

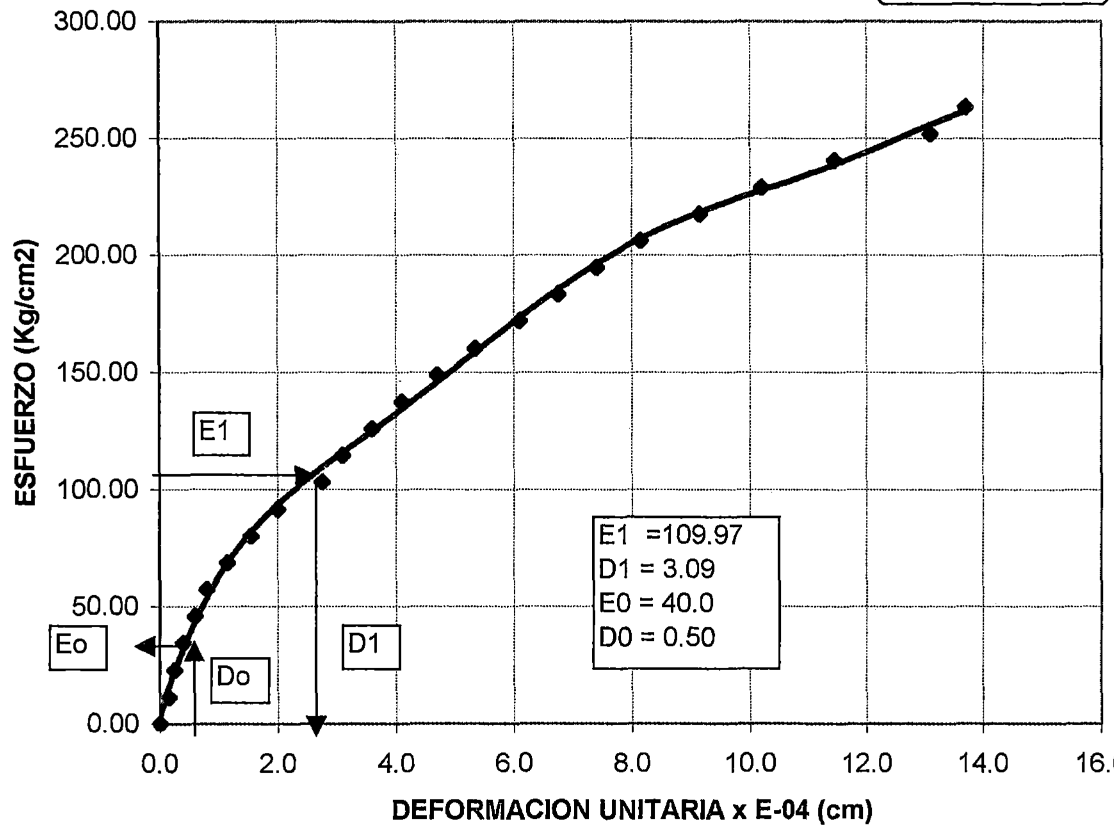
GRAFICO 8.5



Esfuerzo	Def.Unit
0.00	0.00
11.45	0.15
22.91	0.25
34.36	0.40
45.82	0.60
57.27	0.80
68.73	1.15
80.18	1.55
91.64	2.00
103.09	2.75
114.55	3.10
126.00	3.60
137.46	4.10
148.91	4.70
160.37	5.35
171.82	6.10
183.28	6.75
194.73	7.40
206.18	8.15
217.64	9.15
229.09	10.20
240.55	11.45
252.00	13.10
263.46	13.70

MÓDULO ELÁSTICO ESTÁTICO CONCRETO PATRÓN
RELACIÓN AGUA CEMENTO 0.65

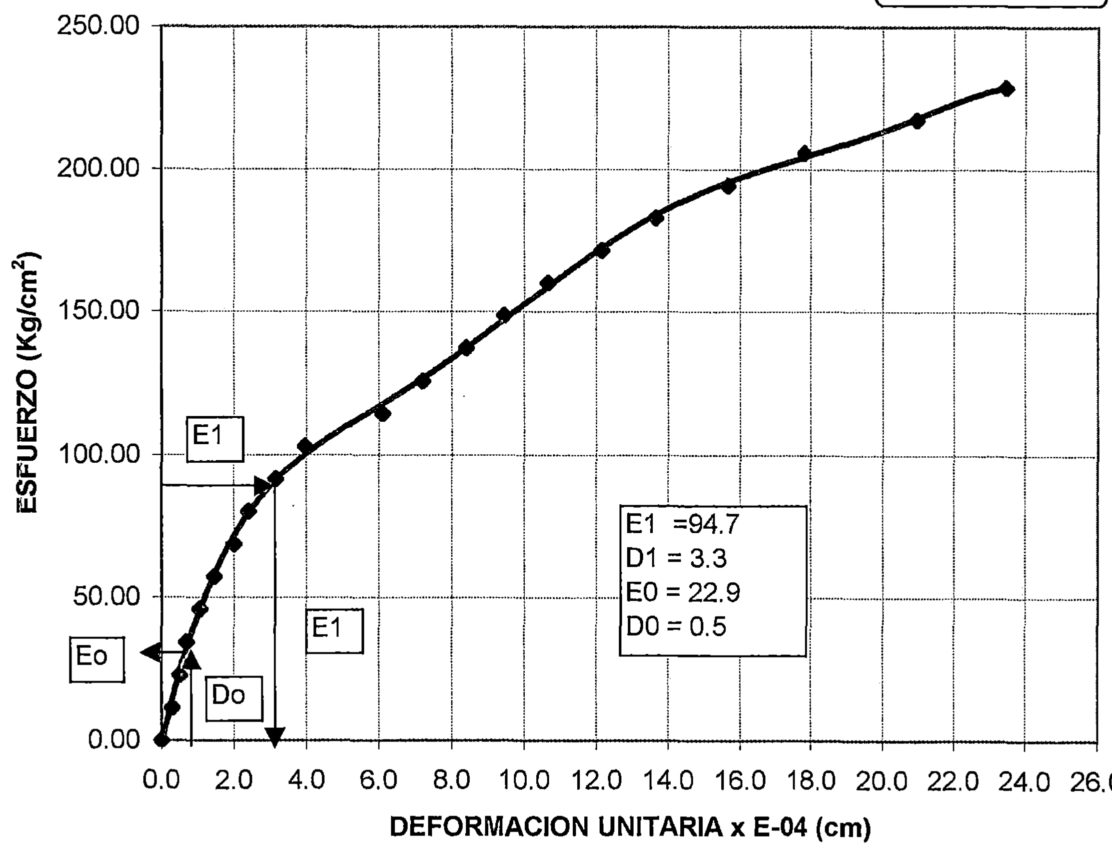
GRAFICO 8.6



Esfuerzo	Def.Unit
0.00	0.00
11.44	0.30
22.88	0.50
34.32	0.70
45.76	1.05
57.20	1.45
68.64	2.00
80.08	2.40
91.51	3.15
102.95	3.95
114.39	6.10
125.83	7.20
137.27	8.40
148.71	9.45
160.15	10.65
171.59	12.15
183.03	13.65
194.47	15.65
205.91	17.80
217.35	20.95
228.79	23.45

MODULO ELASTICO ESTATICO CONCRETO PATRON
RELACION AGUA CEMENTO 0.70

GRAFICO 8.7



8.1.2.4 ENSAYO DE RESISTENCIA A LA FLEXIÓN EN VIGA (kg/cm²)

RELACIÓN AGUA CEMENTO 0.60

N°Dias	Carga (Kg)	Base (cm)	Altura (cm)	Luz (cm)	Modulo (Kg/cm²)	Promedio (Kg/cm²)
28	2500	15.00	15.10	60.00	43.86	41.15
28	2480	15.00	15.10	60.00	43.51	
28	2070	15.10	15.10	60.00	36.07	

RELACION AGUA CEMENTO 0.65

N°Dias	Carga (Kg)	Base (cm)	Altura (cm)	Luz (cm)	Modulo (Kg/cm²)	Promedio (Kg/cm²)
28	2100	15.00	15.00	60.00	37.33	38.35
28	2160	15.10	15.10	60.00	37.64	
28	2300	15.10	15.10	60.00	40.08	

RELACION AGUA CEMENTO 0.70

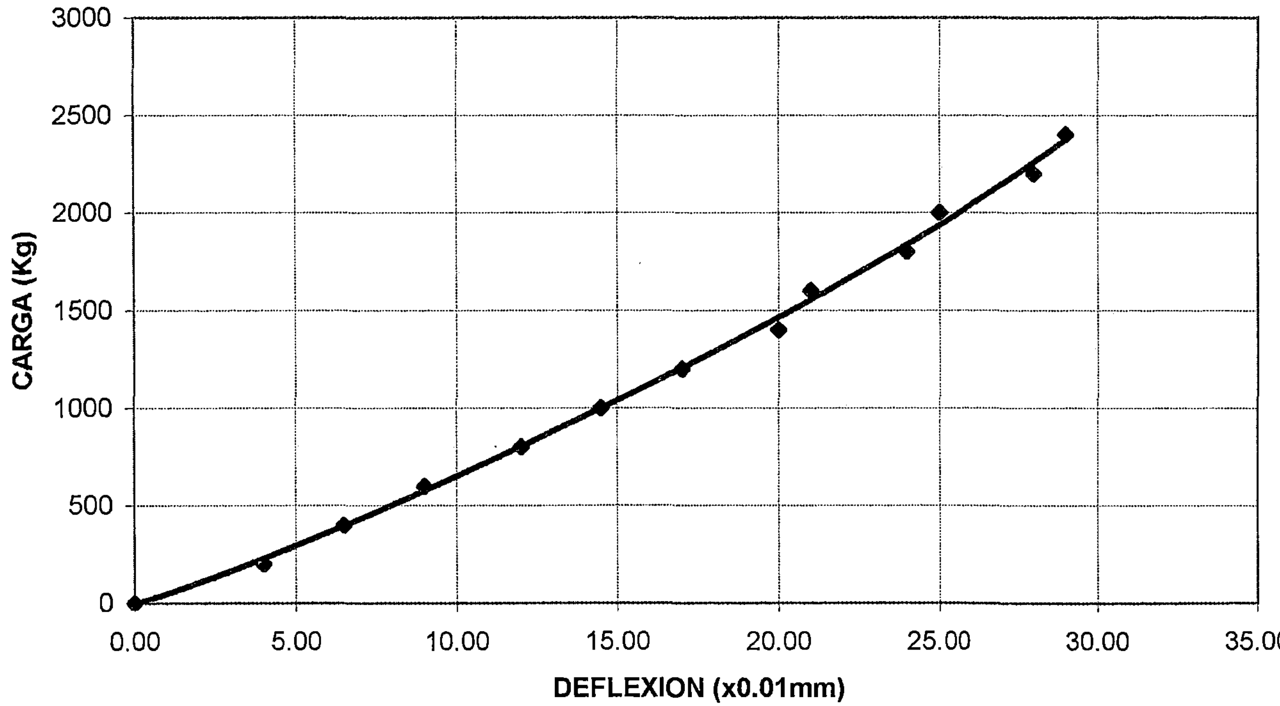
N°Dias	Carga (Kg)	Base (cm)	Altura (cm)	Luz (cm)	Modulo (Kg/cm²)	Promedio (Kg/cm²)
28	2020	15.10	15.10	60.00	35.20	35.54
28	1960	15.10	15.00	60.00	34.61	
28	2070	15.00	15.00	60.00	36.80	

RESISTENCIA A LA FLEXIÓN EN VIGAS
CONCRETO PATRÓN
a/c=0.60

GRAFICO 8.8

Carga (Kg)	Deflexión (x0.01mm)
0	0.00
200	4.00
400	6.50
600	9.00
800	12.00
1000	14.50
1200	17.00
1400	20.00
1600	21.00
1800	24.00
2000	25.00
2200	28.00
2400	29.00

Fluencia	**2500**

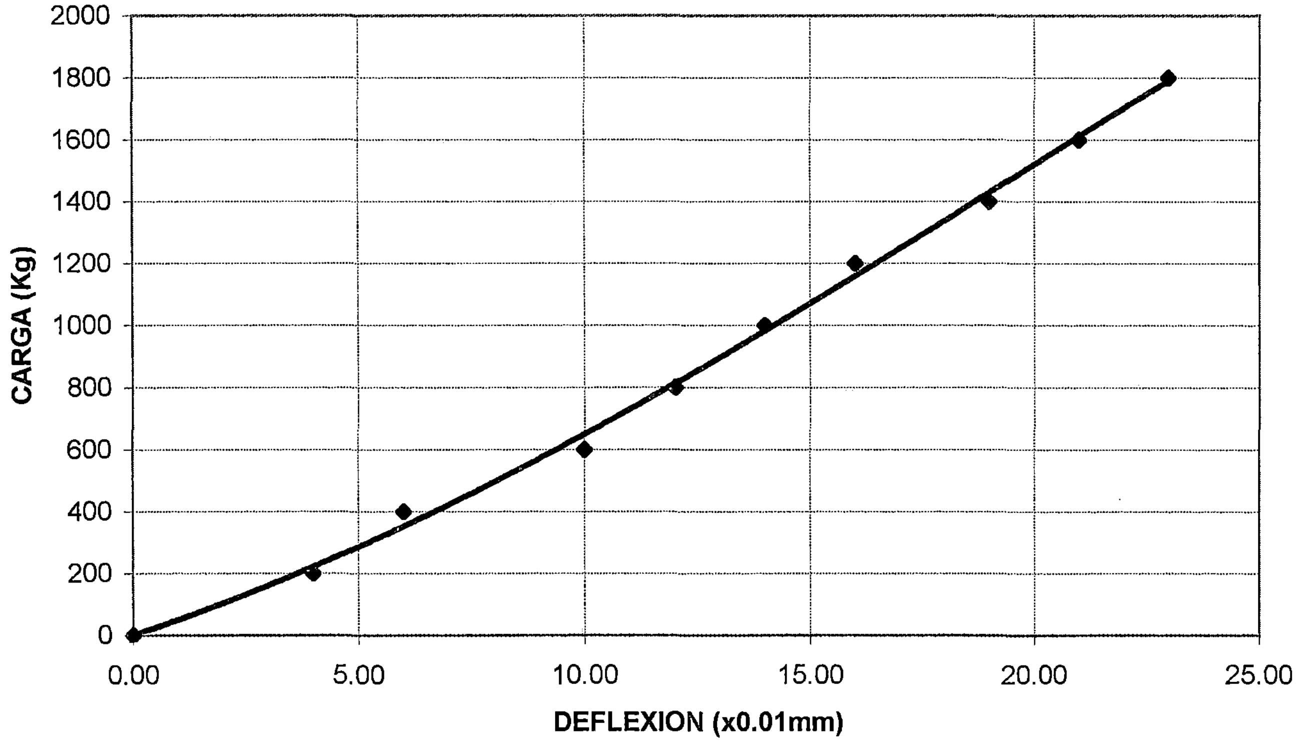


RESISTENCIA A LA FLEXIÓN EN VIGAS
CONCRETO PATRÓN
a/c=0.65

GRAFICO 8.9

Carga (Kg)	Deflexión (x0.01mm)
0	0.00
200	4.00
400	6.00
600	10.00
800	12.00
1000	14.00
1200	16.00
1400	19.00
1600	21.00
1800	23.00

Fluencia	2070

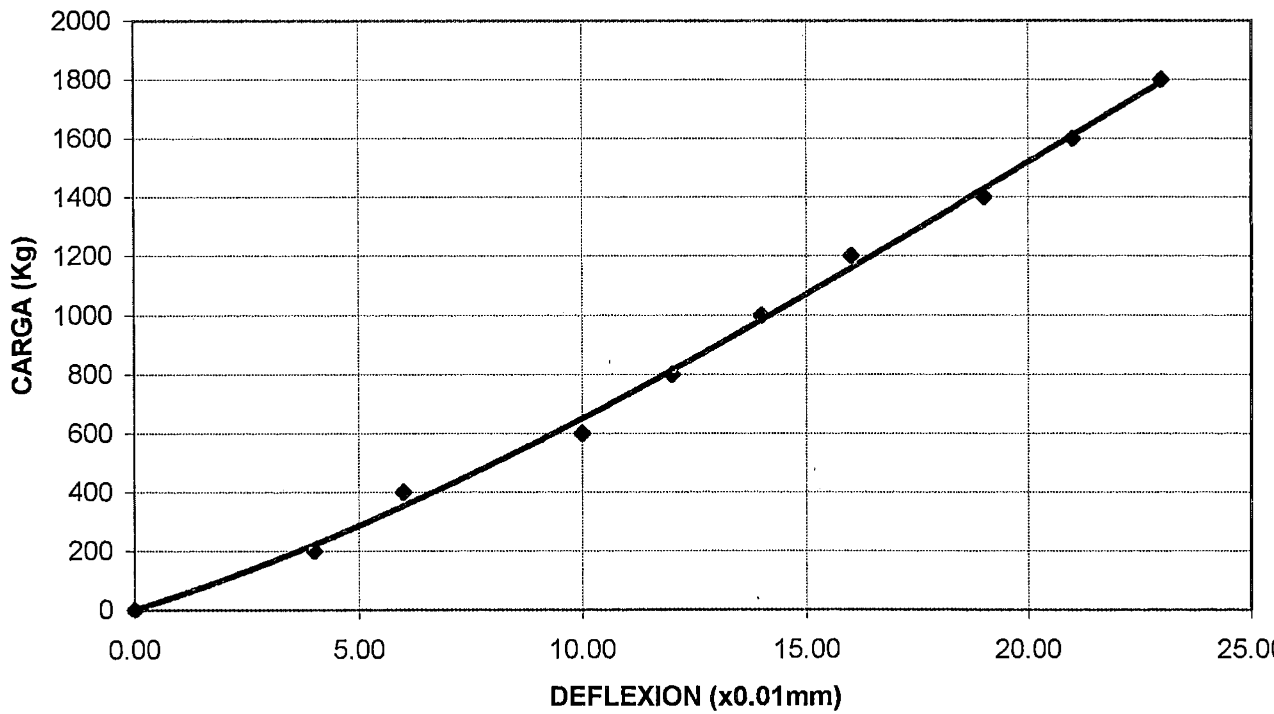


RESISTENCIA A LA FLEXIÓN EN VIGA
CONCRETO PATRÓN
a/c=0.70

GRAFICO 8.10

Carga (Kg)	Deflexión (x0.01mm)
0	0.00
200	3.00
400	6.00
600	9.50
800	12.00
1000	15.00
1200	17.00
1400	19.00
1600	21.00
1800	23.00
2000	25.00
2200	27.00

Fluencia	2450



8.1.2.5 ENSAYO DE RESISTENCIA AL IMPACTO

RELACION AGUA CEMENTO 0.60

N° Días	Altura		Diámetro		N° Golpes	
	H	Prom	D	Prom	G	Prom
28	6.90		15.00		70	
28	6.60	6.63	15.00	15.05	60	63
28	6.40		15.15		60	
42	6.60		15.00		62	
42	6.50	6.60	15.00	15.00	64	66
42	6.70		15.00		72	

RELACION AGUA CEMENTO 0.65

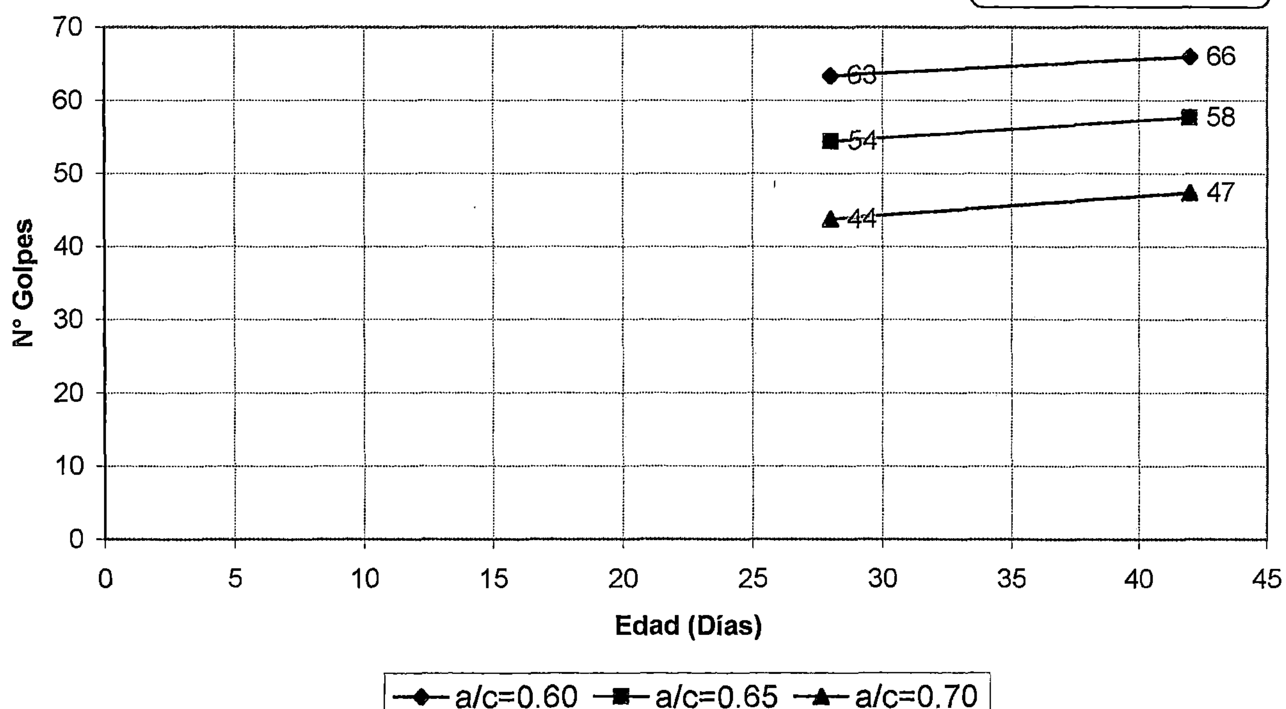
N° Días	Altura		Diámetro		N° Golpes	
	H	Prom	D	Prom	G	Prom
28	6.50		14.94		45	
28	6.50	6.53	14.94	14.96	60	54
28	6.60		15.00		58	
42	6.63		14.90		61	
42	6.45	6.54	15.00	14.97	50	58
42	6.55		15.00		62	

RELACIÓN AGUA CEMENTO 0.70

N° Días	Altura		Diámetro		N° Golpes	
	H	Prom	D	Prom	G	Prom
28	6.60		14.98		45	
28	6.40	6.50	14.96	14.97	35	44
28	6.50		14.98		51	
42	6.65		14.80		56	
42	6.50	6.52	14.95	14.88	41	47
42	6.40		14.90		45	

ENSAYO DE RESISTENCIA ALIMPACTO
GRÁFICO COMPARATIVO PARA DIFERENTES RELACIONES DE A/C

GRÁFICO 8.11



8.2 ENSAYOS EN EL CONCRETO CON FIBRAS DE ACERO

8.2.1 ENSAYOS EN EL CONCRETO FRESCO

8.2.1.1 ENSAYO DE ASENTAMIENTO (plg)

- **RELACIÓN AGUA CEMENTO 0.60**

Dosificación (Kg/m^3)	Asentamiento
35	3 1/2"
45	3 1/4"
55	3"

- **RELACIÓN AGUA CEMENTO 0.65**

Dosificación (Kg/m^3)	Asentamiento
35	3 3/4"
45	3 1/2"
55	3 1/4"

- **RELACIÓN AGUA CEMENTO 0.70**

Dosificación (Kg/m^3)	Asentamiento
35	4"
45	3 3/4"
55	3 1/2"

8.2.1.2 ENSAYO DE EXUDACIÓN (%)

- **RELACIÓN AGUA CEMENTO 0.60**

Dosificación (Kg/m^3)	Muestra	%Exudac	Promedio
35	E-1	2.48	2.50
	E-2	2.53	
45	E-1	2.50	2.42
	E-2	2.35	
55	E-1	2.35	2.30
	E-2	2.25	

- **RELACIÓN AGUA CEMENTO 0.65**

Dosificación (Kg/m^3)	Muestra	%Exudac	Promedio
35	E-1	2.73	2.70
	E-2	2.66	
45	E-1	2.57	2.50
	E-2	2.42	
55	E-1	2.44	2.43
	E-2	2.41	

- **RELACIÓN AGUA CEMENTO 0.70**

Dosificación (Kg/m^3)	Muestra	%Exudac	Promedio
35.00	E-1	2.87	2.79
	E-2	2.72	
45.00	E-1	2.61	2.53
	E-2	2.46	
55.00	E-1	2.51	2.44
	E-2	2.37	

8.2.1.3 ENSAYO DE PESO UNITARIO COMPACTADO (kg/m^3)

- **RELACIÓN AGUA CEMENTO 0.60**

Dosificación (Kg/m^3)	Peso Unitario Compactado (Kg/m^3)
35	2446.43
45	2450.00
55	2464.29

- **RELACIÓN AGUA CEMENTO 0.65**

Dosificación (Kg/m^3)	Peso Unitario Compactado (Kg/m^3)
35	2478.57
45	2485.71
55	2492.86

- **RELACIÓN AGUA CEMENTO 0.70**

Dosificación (Kg/m^3)	Peso Unitario Compactado (Kg/m^3)
35.00	2457.14
45.00	2492.86
55.00	2500.00

8.2.1.4 ENSAYO DE TIEMPO DE FRAGUADO (Hr:min)

> - **RELACIÓN AGUA CEMENTO 0.60**

Dosificación (Kg/m^3)	Sección Plg^2	Tiempo Hr	Fuerza Lbs	Resistencia (Lbs/plg^2)	Fragua Inicial	Fragua Final
35		00:00		0.00	4:15	5:20
	0.99402	03:00	170	171.02		
	0.51848	03:30	145	279.66		
	0.24850	04:15	125	503.02		
	0.07669	04:30	150	1955.93		
	0.04908	05:00	175	3565.61		
	0.02761	05:36	200	7243.75		
45		00:00		0.00	4:11	5:16
	0.99402	03:12	125	125.75		
	0.51848	03:36	170	327.88		
	0.24850	04:12	125	503.02		
	0.07669	04:42	165	2151.52		
	0.04908	05:12	170	3463.73		
	0.02761	05:42	200	7243.75		
55		00:00		0.00	4:06	5:10
	0.99402	03:30	130	130.78		
	0.51848	03:54	150	289.31		
	0.24850	04:24	150	603.62		
	0.07669	04:48	135	1760.33		
	0.04908	05:12	200	4074.98		
	0.02761	05:48	200	7243.75		

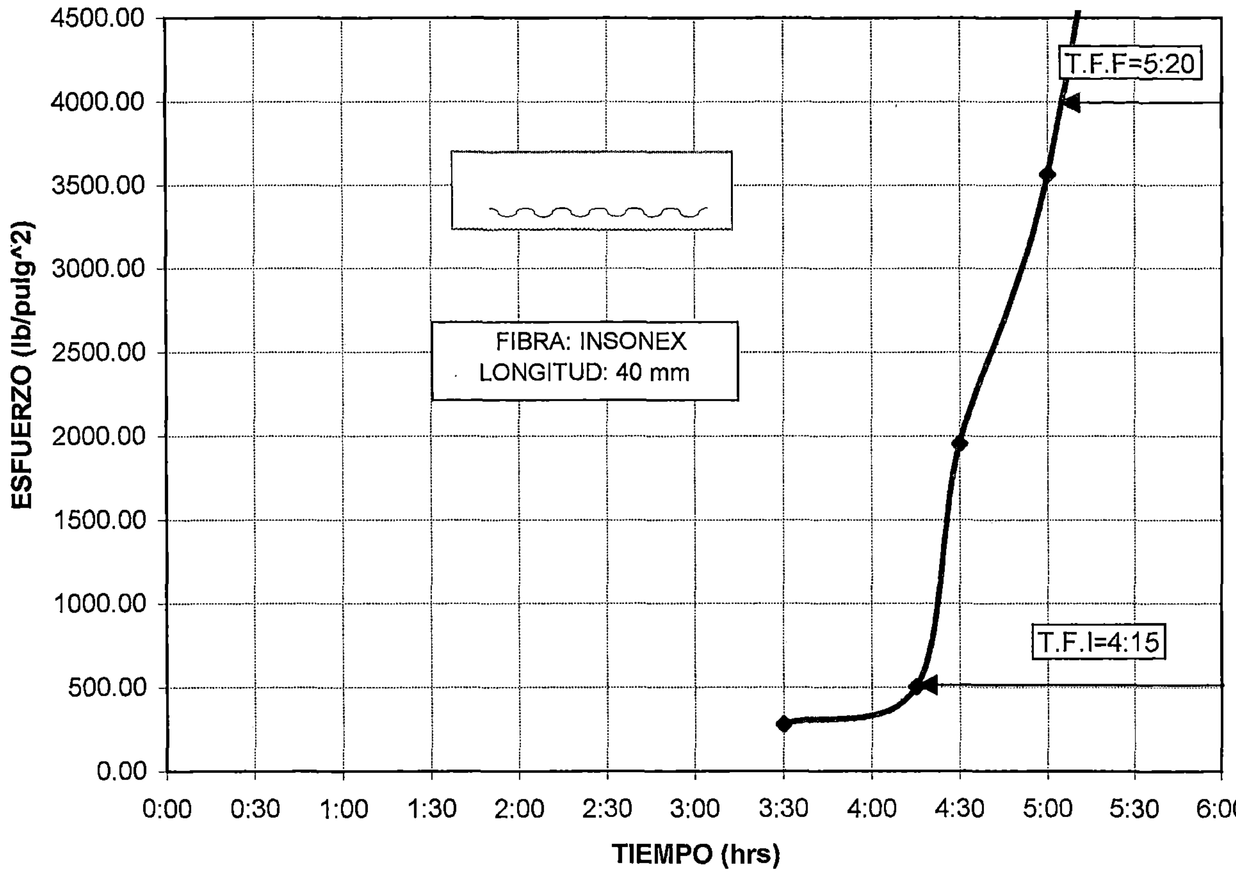
CONCRETO CON FIBRA

DOSIFICACION: 35 kg/m^3 DE CONCRETO

ENSAYO DE TIEMPO DE FRAGUADO

A/C = 0.60

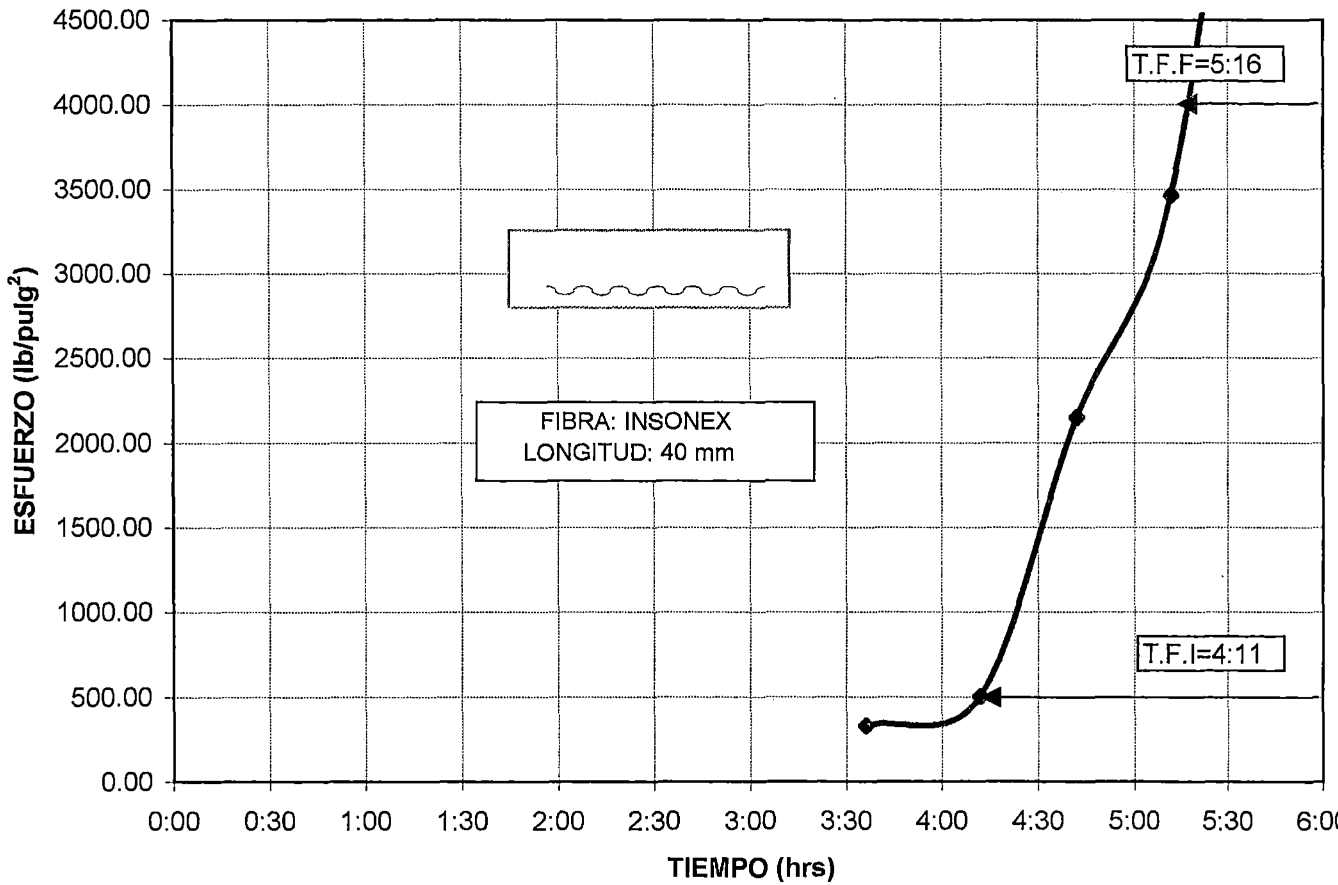
GRAFICO 8.12



CONCRETO CON FIBRA
DOSIFICACION: 45 kg/m3 DE CONCRETO

ENSAYO DE TIEMPO DE FRAGUADO
A/C = 0.60

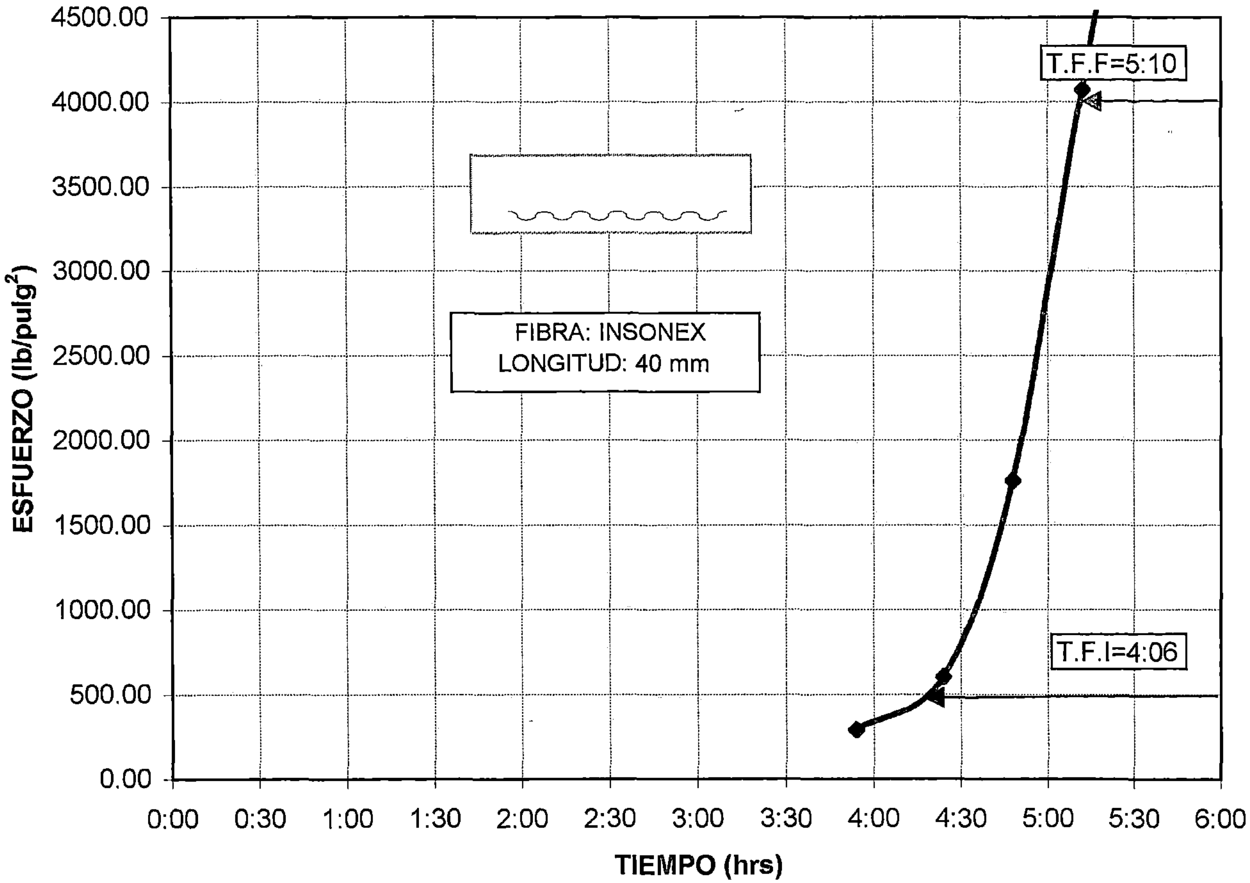
GRAFICO 8.13



CONCRETO CON FIBRA
DOSIFICACION: 55 kg/m3 DE CONCRETO

ENSAYO DE TIEMPO DE FRAGUADO
A/C = 0.60

GRAFICO 8.14



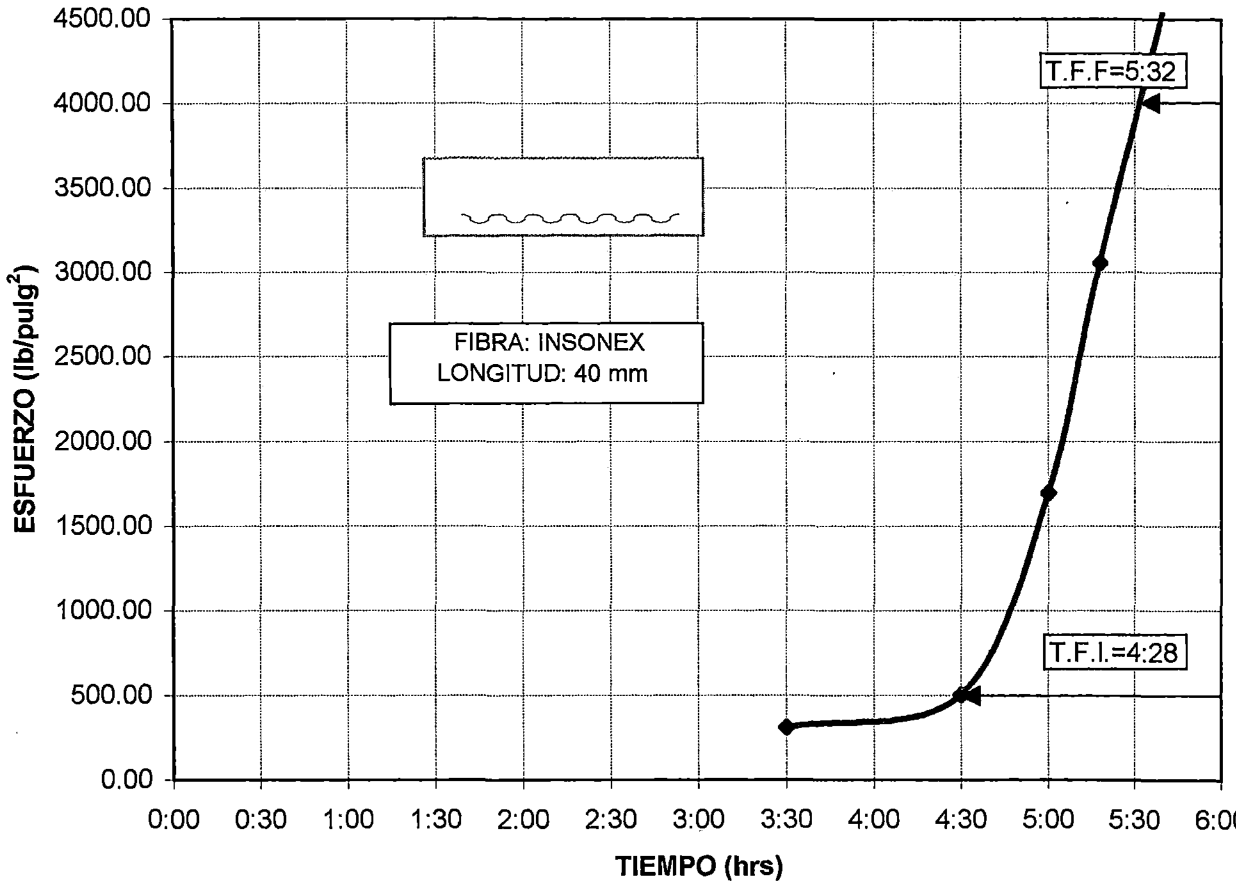
➢ RELACIÓN AGUA CEMENTO 0.65

Dosificación (Kg/m^3)	Sección Plg^2	Tiempo Hr	Fuerza Lbs	Resistencia (Lbs/plg^2)	Fragua Inicial	Fragua Final
35		00:00		0.00	4:28	5:32
	0.99402	03:00	140	140.84		
	0.51848	03:30	160	308.59		
	0.24850	04:30	125	503.02		
	0.07669	05:00	130	1695.14		
	0.04908	05:18	150	3056.23		
	0.02761	05:48	200	7243.75		
45		00:00		0.00	4:25	5:30
	0.99402	03:30	145	145.87		
	0.51848	03:48	140	270.02		
	0.24850	04:36	130	523.14		
	0.07669	05:00	160	2086.32		
	0.04908	05:24	130	2648.74		
	0.02761	05:48	200	7243.75		
55		00:00		0.00	4:20	5:23
	0.99402	03:30	120	120.72		
	0.51848	03:48	130	250.73		
	0.24850	04:25	125	503.02		
	0.07669	05:00	175	2281.91		
	0.04908	05:25	200	4074.98		
	0.02761	06:00	200	7243.75		

CONCRETO CON FIBRA
DOSIFICACION: 35 kg/m3 DE CONCRETO

ENSAYO DE TIEMPO DE FRAGUADO
A/C = 0.65

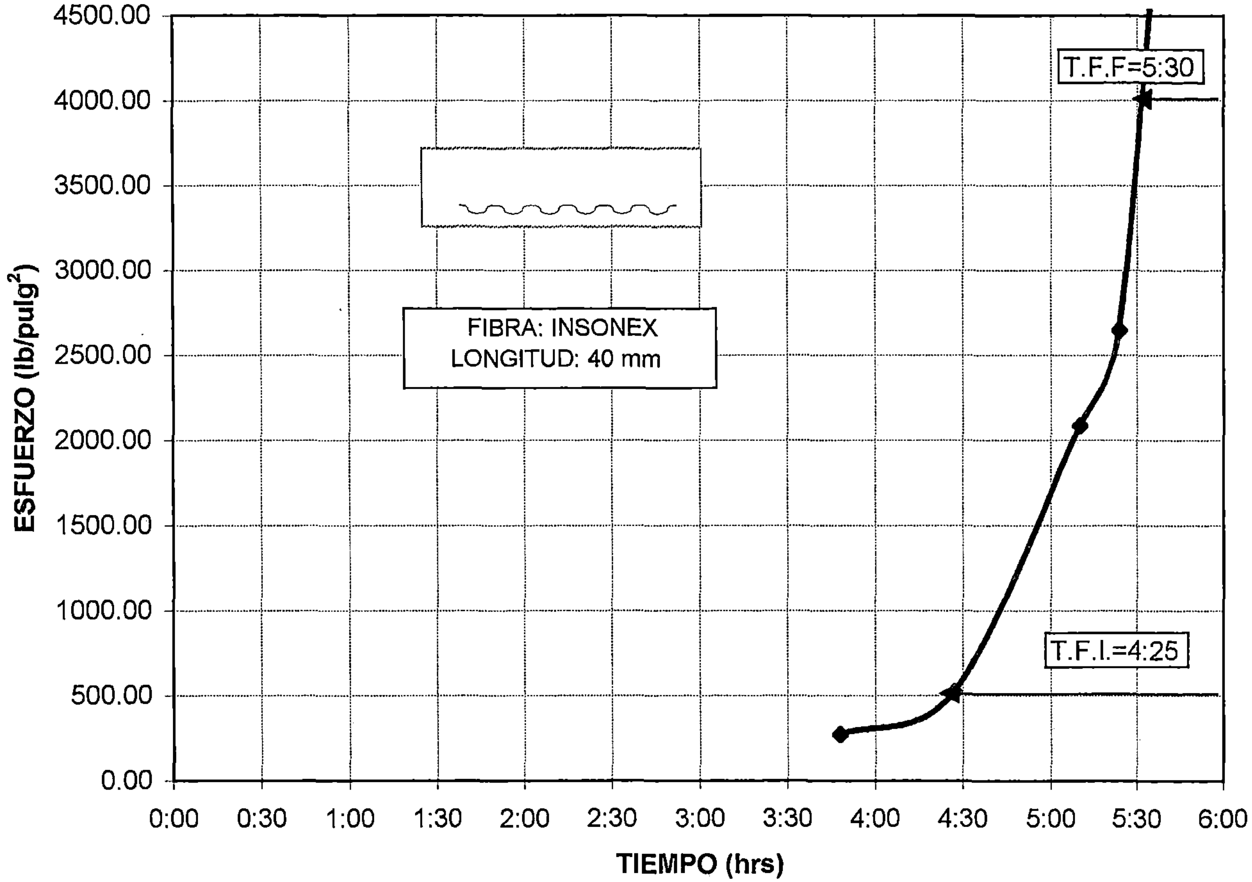
GRAFICO 8.15



CONCRETO CON FIBRA
DOSIFICACION: 45 kg/m3 DE CONCRETO

ENSAYO DE TIEMPO DE FRAGUADO
A/C = 0.65

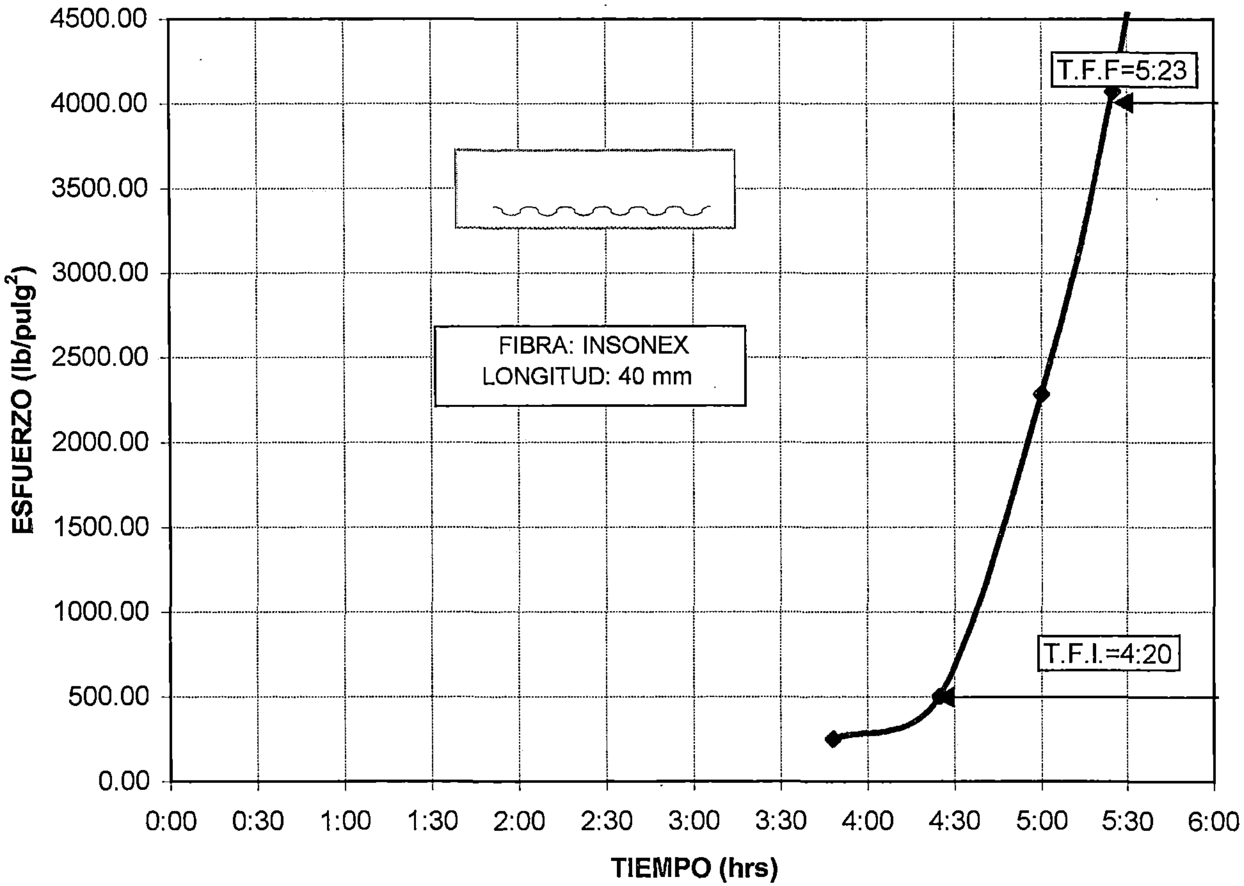
GRAFICO 8.16



CONCRETO CON FIBRA
DOSIFICACION: 55 kg/m3 DE CONCRETO

ENSAYO DE TIEMPO DE FRAGUADO
A/C = 0.65

GRAFICO 8.17



➢ **RELACIÓN AGUA CEMENTO 0.70**

Dosificación (Kg/m^3)	Sección Plg^2	Tiempo Hr	Fuerza Lbs	Resistencia (Lbs/plg^2)	Fragua Inicial	Fragua Final
35		00:00		0.00	4:30	5:42
	0.99402	03:00	120	120.72		
	0.51848	03:30	150	289.31		
	0.24850	04:32	125	503.02		
	0.07669	04:48	160	2086.32		
	0.04908	05:45	200	4074.98		
	0.02761	05:48	170	6157.19		
45		00:00		0.00	4:27	5:36
	0.99402	03:30	160	160.96		
	0.51848	04:00	160	308.59		
	0.24850	04:30	125	503.02		
	0.07669	05:00	130	1695.14		
	0.04908	05:40	200	4074.98		
	0.02761	06:00	190	6881.56		
55		00:00		0.00	4:24	5:25
	0.99402	03:30	130	130.78		
	0.51848	04:00	130	250.73		
	0.24850	04:25	125	503.02		
	0.07669	05:12	130	1695.14		
	0.04908	05:28	200	4074.98		
	0.02761	06:00	180	6519.38		

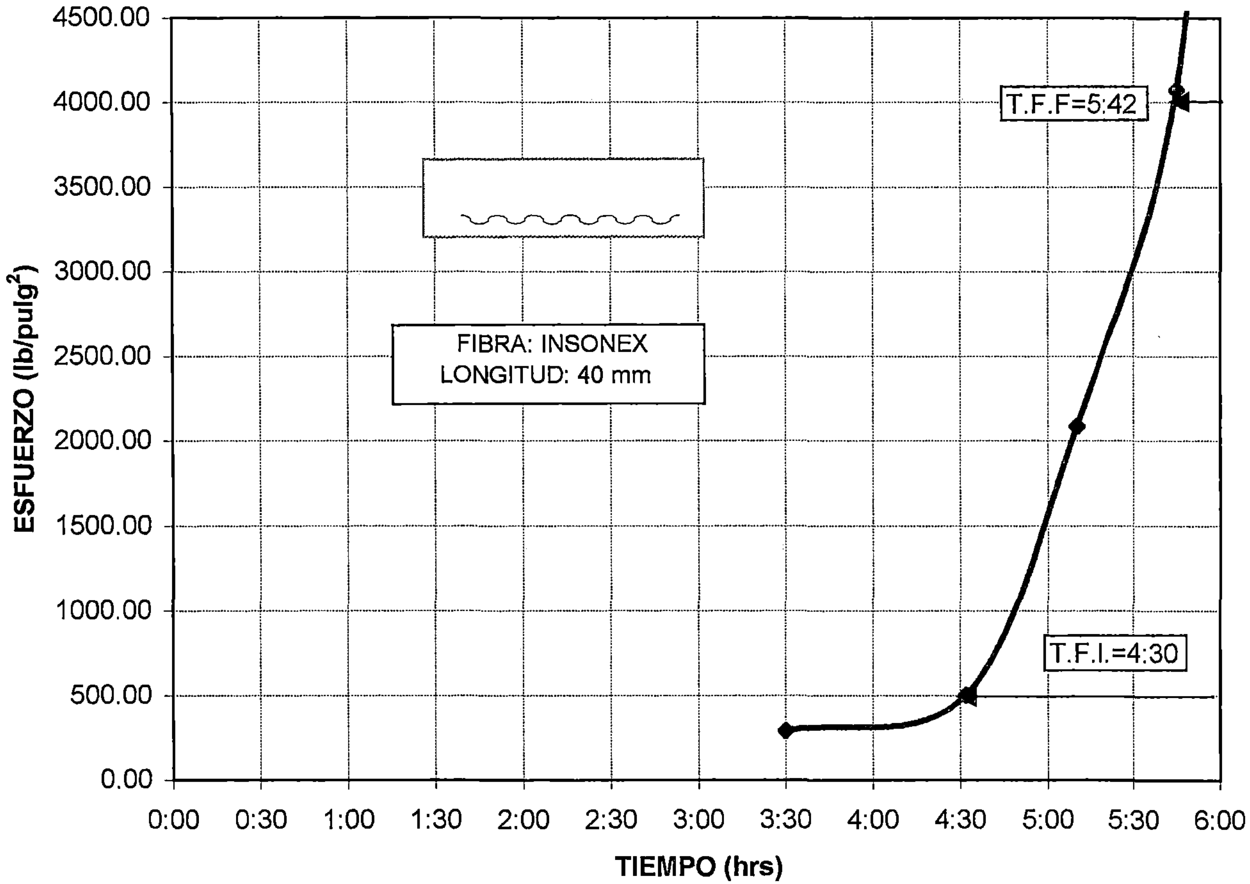
CONCRETO CON FIBRA

DOSIFICACION: 35 kg/m^3 DE CONCRETO

ENSAYO DE TIEMPO DE FRAGUADO

A/C = 0.70

GRAFICO 8.18



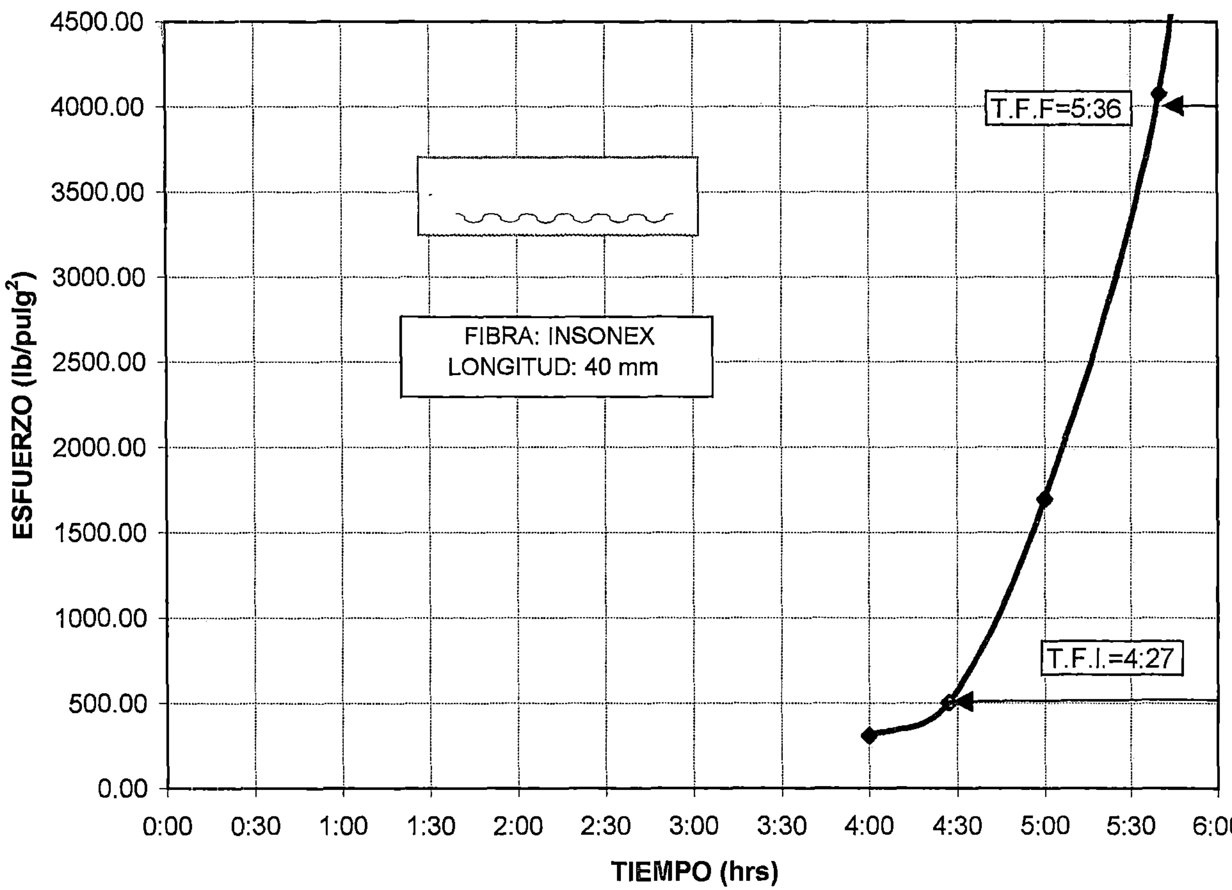
CONCRETO CON FIBRA

DOSIFICACION: 45 kg/m^3 DE CONCRETO

ENSAYO DE TIEMPO DE FRAGUADO

A/C = 0.70

GRAFICO 8.19



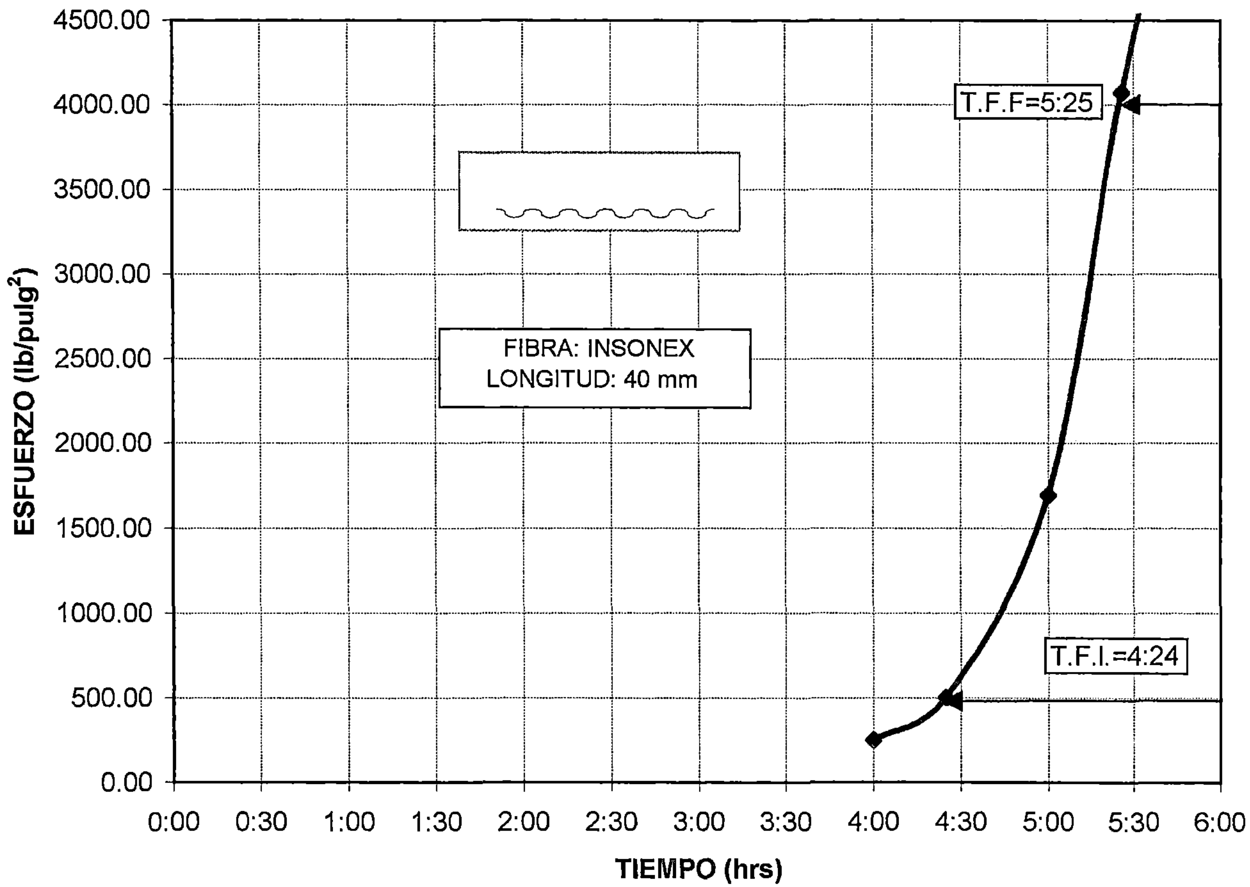
CONCRETO CON FIBRA

DOSIFICACION: 55 kg/m^3 DE CONCRETO

ENSAYO DE TIEMPO DE FRAGUADO

A/C = 0.70

GRAFICO 8.20



8.2.1.5 ENSAYO DE FLUIDEZ (%)

- **RELACIÓN AGUA CEMENTO 0.60**

Dosifición (Kg/m^3)	Fluidez (%)
35	102.76
45	98.82
55	93.24

- **RELACIÓN AGUA CEMENTO 0.65**

Dosificación (Kg/m^3)	Fluidez (%)
35	106.04
45	103.41
55	96.19

- **RELACIÓN AGUA CEMENTO 0.70**

Dosificación (Kg/m^3)	Fluidez (%)
35	107.35
45	102.43
55	97.51

8.2.1.6 ENSAYO DE CONTENIDO DE AIRE (%)

- **RELACIÓN AGUA CEMENTO 0.60**

Dosificación (Kg/m^3)	Contenido de Aire (%)
35	1.90
45	2.10
55	2.14

- **RELACIÓN AGUA CEMENTO 0.65**

Dosifición (Kg/m^3)	Contenido de Aire (%)
35	1.70
45	1.90
55	2.05

- **RELACIÓN AGUA CEMENTO 0.70**

Dosificación (Kg/m^3)	Contenido de Aire (%)
35.00	1.50
45.00	1.80
55.00	1.90

8.2.2 ENSAYOS EN EL CONCRETO ENDURECIDO

8.2.2.1 ENSAYO DE RESISTENCIA A LA COMPRESIÓN (kg/cm^2)

- **RELACIÓN AGUA CEMENTO 0.60**
  - **DOSIFICACIÓN DE LA FIBRA 35 Kg/m^3 DE CONCRETO**

N° Dias	Carga (Kg)	Diametro (cm)	Resistencia (Kg/cm^2)	Promedio (Kg/cm^2)
7	36700	15.00	207.68	207.37
7	35600	14.94	203.08	
7	37200	14.97	211.35	
14	43400	15.05	243.96	249.12
14	43800	15.00	247.86	
14	45400	15.04	255.55	
28	51100	14.80	297.03	312.73
28	54500	14.90	312.56	
28	57000	14.80	331.33	
28	54800	14.85	316.40	
28	53000	14.80	308.08	
28	53500	14.80	310.99	
42	55800	15.00	315.76	334.47
42	57800	14.80	335.98	
42	60500	14.80	351.67	

- **DOSIFICACIÓN DE LA FIBRA 45 Kg/m³ DE CONCRETO**

N° Días	Carga (Kg)	Diámetro (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
7	37500	14.94	213.91	212.01
7	38000	14.98	215.61	
7	36200	14.94	206.50	
14	44900	15.07	251.73	252.78
14	45400	15.00	256.91	
14	44600	15.08	249.71	
28	53300	14.84	308.15	313.06
28	52900	14.96	300.96	
28	52500	14.80	305.17	
28	52100	14.85	300.81	
28	58000	14.80	337.14	
28	56100	14.80	326.10	
42	59600	14.80	346.44	335.67
42	54800	14.94	312.60	
42	61000	14.94	347.97	

- **DOSIFICACIÓN DE LA FIBRA 55 Kg/m³ DE CONCRETO**

N° Dias	Carga (Kg)	Diámetro (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
7	37400	15	211.64	224.43
7	39300	14.89	225.69	
7	41700	15	235.97	
14	42800	15.06	240.27	259.28
14	49400	14.94	281.80	
14	44600	14.90	255.78	
28	52000	14.80	302.27	315.01
28	55600	14.90	318.87	
28	54700	14.80	317.96	
28	51400	14.82	297.97	
28	55800	15.00	315.76	
28	59200	14.95	337.25	
42	55000	14.93	314.16	336.80
42	60200	14.90	345.25	
42	61200	14.90	350.98	

- **RELACIÓN AGUA CEMENTO 0.65**
  - **DOSIFICACIÓN DE LA FIBRA 35 Kg/m³ DE CONCRETO**

N° Días	Carga (Kg)	Diámetro (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
7	29700	14.80	172.64	156.88
7	27000	14.82	156.52	
7	25000	15.00	141.47	
14	36800	14.90	211.05	210.34
14	36100	14.81	209.56	
14	36200	14.80	210.42	
28	43400	15.02	244.94	259.77
28	44000	14.80	255.76	
28	43400	14.80	252.28	
28	48400	14.80	281.34	
28	47200	14.80	274.36	
28	43000	14.80	249.95	
42	50800	14.90	291.34	299.46
42	50000	14.90	286.75	
42	55100	14.80	320.29	

  - **DOSIFICACIÓN DE LA FIBRA 45 Kg/m3 DE CONCRETO**

N° Días	Carga (Kg)	Diámetro (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
7	29300	15.20	161.47	164.73
7	31600	15.20	174.14	
7	28400	15.10	158.59	
14	35300	14.80	205.19	211.88
14	36300	14.81	210.72	
14	37800	14.80	219.72	
28	43000	14.95	244.96	264.00
28	48800	14.90	279.87	
28	45400	14.90	260.37	
28	43600	14.80	253.44	
28	48400	14.81	280.96	
28	46600	14.98	264.41	
42	57200	14.90	328.04	305.40
42	55000	14.80	319.70	
42	47000	14.93	268.46	

- DOSIFICACIÓN DE LA FIBRA 55 Kg/m³ DE CONCRETO

N° Días	Carga (Kg)	Diámetro (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
7	32000	14.92	183.03	167.19
7	28200	14.80	163.92	
7	26600	14.80	154.62	
14	39700	15.00	224.66	219.11
14	36900	14.94	210.49	
14	39000	14.95	222.17	
28	45500	15.00	257.48	269.13
28	42800	14.80	248.79	
28	43800	14.80	254.60	
28	50000	14.80	290.64	
28	50000	15.10	279.21	
28	50200	15.00	284.07	
42	57400	14.90	329.19	311.25
42	56500	15.00	319.72	
42	49800	14.92	284.84	

- **RELACIÓN AGUA CEMENTO 0.70**
- **DOSIFICACIÓN DE LA FIBRA 35 Kg/m³ DE CONCRETO**

N° Días	Carga (Kg)	Diámetro (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
7	25600	14.96	145.64	145.28
7	23800	14.90	136.49	
7	26800	14.90	153.70	
14	35100	14.90	201.30	203.33
14	33900	14.80	197.05	
14	36900	14.90	211.62	
28	44400	15.00	251.25	237.73
28	42200	14.85	243.65	
28	42800	15.05	240.59	
28	41800	14.95	238.12	
28	40600	14.97	230.67	
28	39300	15.01	222.10	
42	48600	14.83	281.36	263.17
42	43400	15.00	245.59	
42	46400	15.00	262.57	

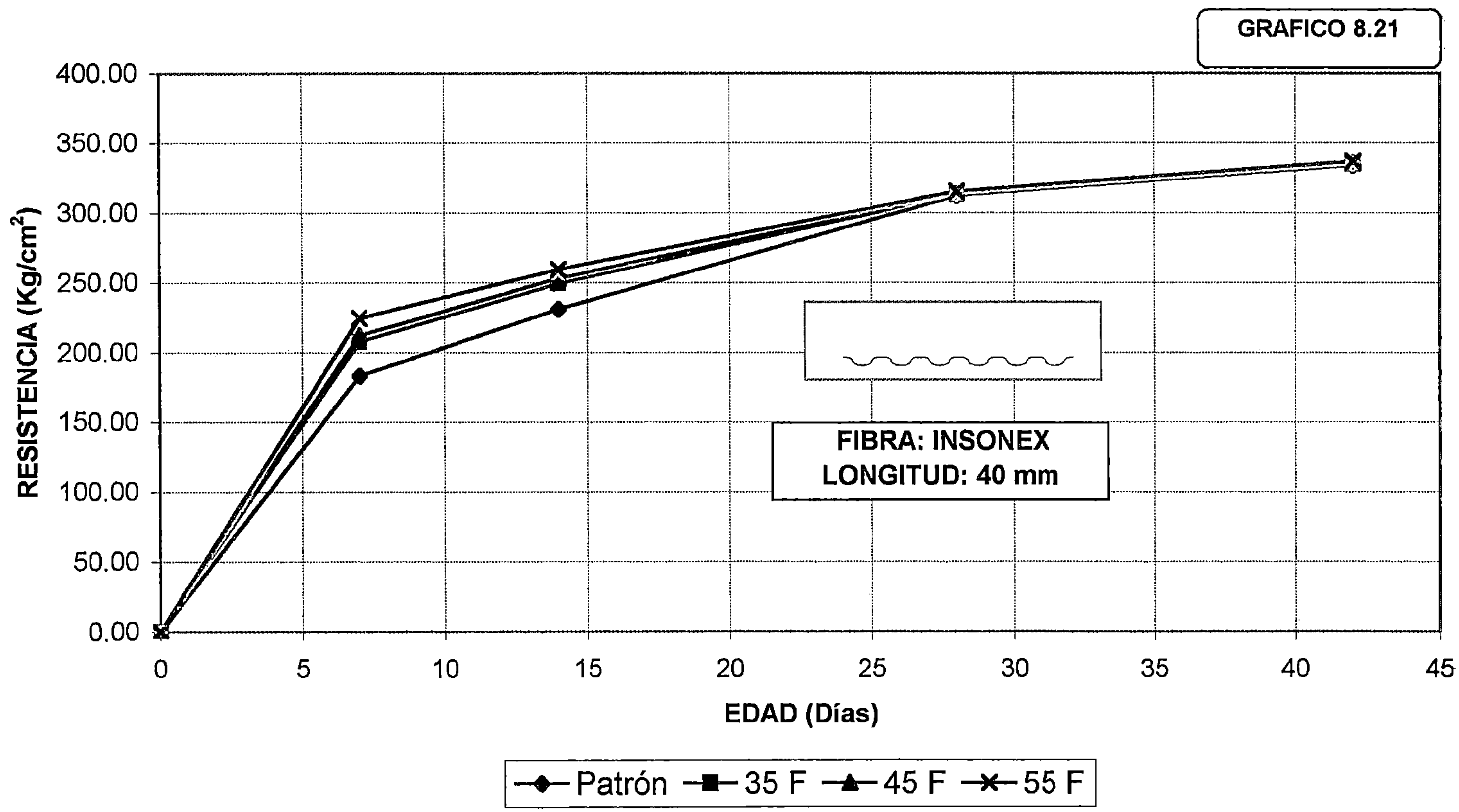
- **DOSIFICACIÓN DE LA FIBRA 45 Kg/m³ DE CONCRETO**

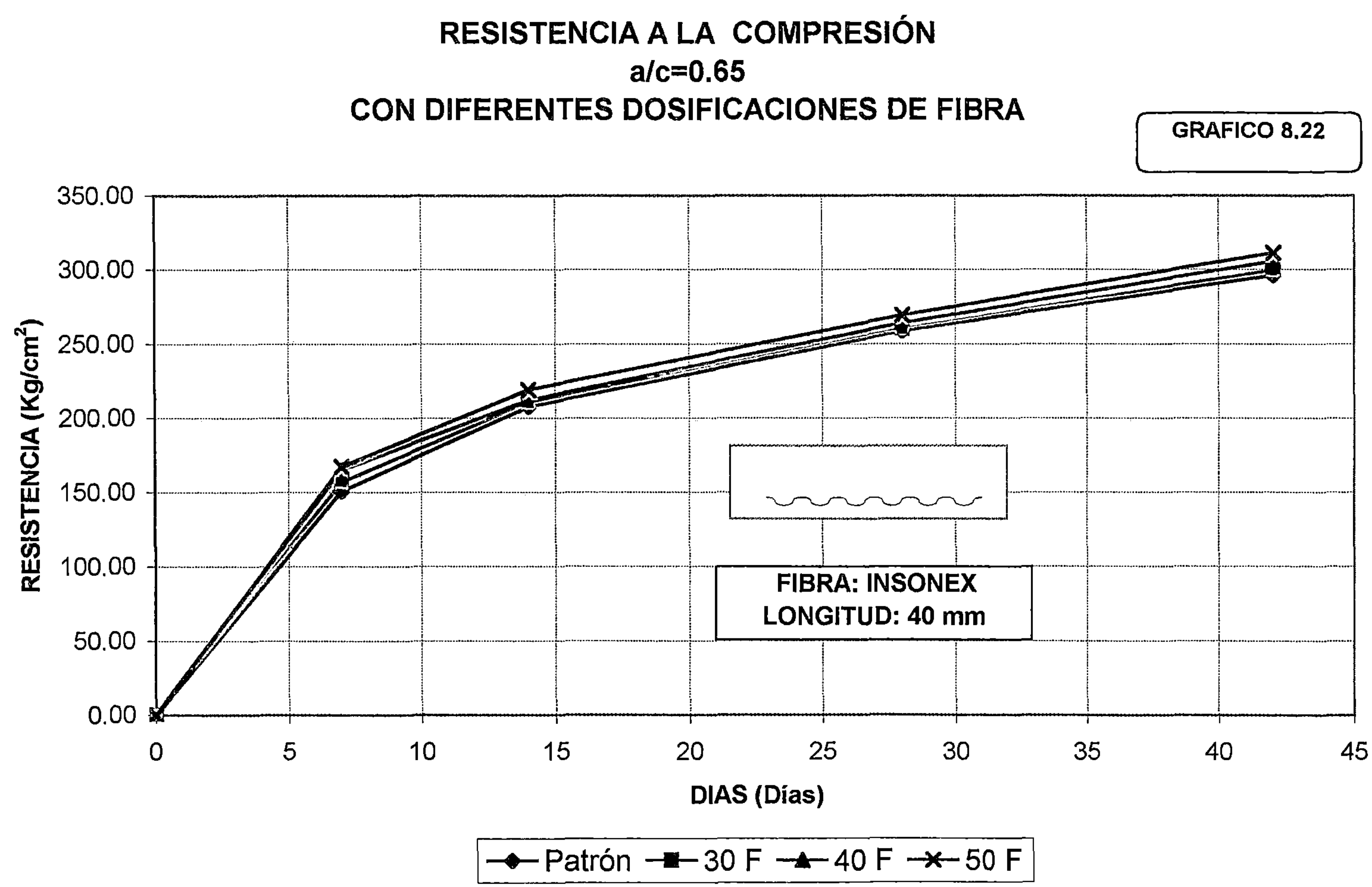
N° Días	Carga (Kg)	Diámetro (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
7	26200	15.00	148.26	148.47
7	25200	14.83	145.89	
7	26200	14.85	151.27	
14	35400	14.80	205.77	207.25
14	35000	14.85	202.08	
14	37800	15.00	213.90	
28	39600	15.13	220.26	239.90
28	40600	15.00	229.75	
28	40000	15.00	226.35	
28	49100	14.90	281.59	
28	45400	14.95	258.63	
28	39800	15.08	222.84	
42	52600	14.80	305.75	267.78
42	42400	14.97	240.90	
42	45000	14.94	256.70	

- **DOSIFICACIÓN DE LA FIBRA 55 Kg/m³ DE CONCRETO**

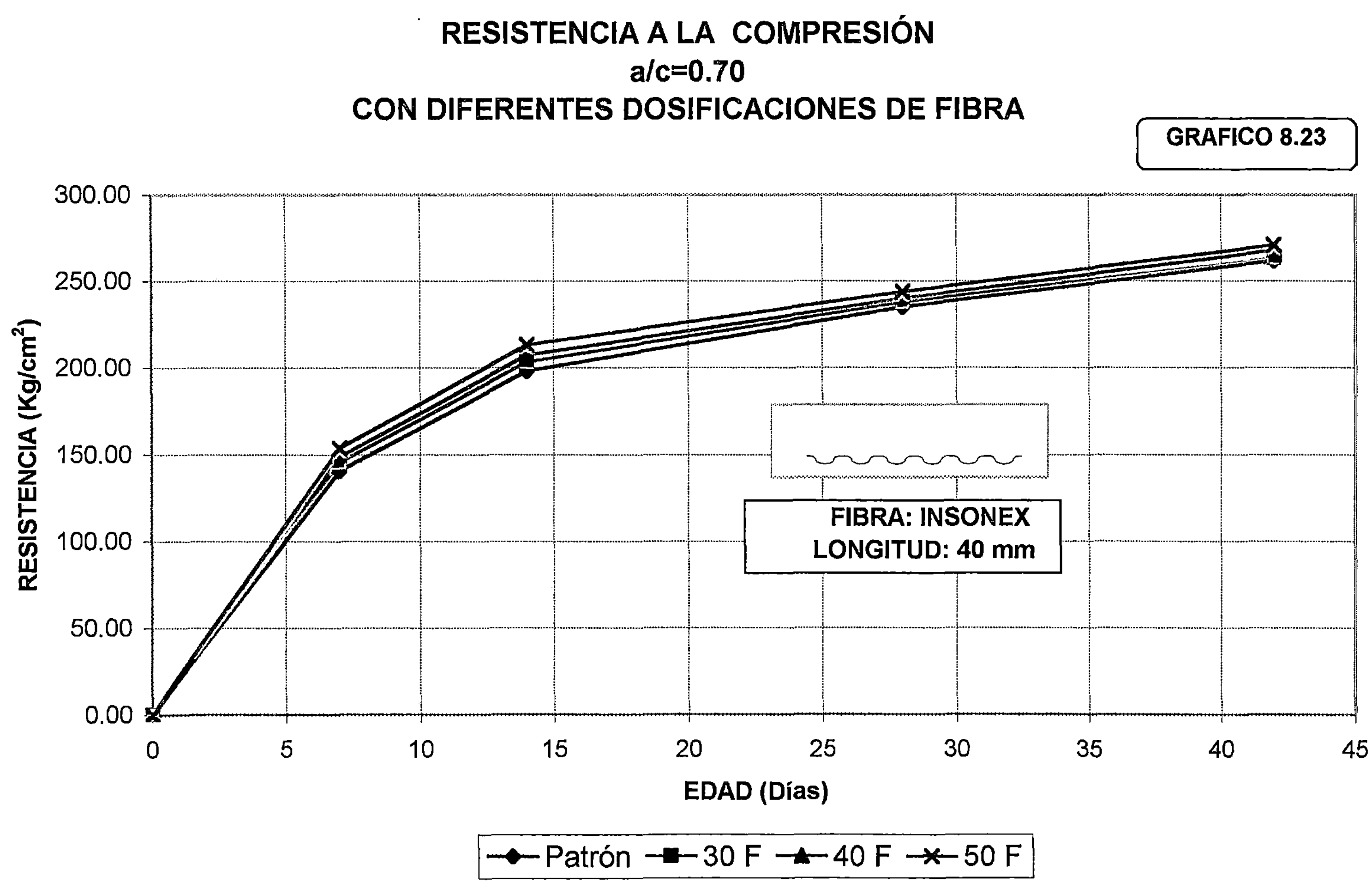
N° Días	Carga (Kg)	Diámetro (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
7	26300	14.94	150.03	153.55
7	28400	14.85	163.97	
7	25400	14.85	146.65	
14	35000	14.95	199.39	203.65
14	34300	14.80	199.38	
14	37000	14.90	212.20	
28	41700	14.90	239.15	243.70
28	41900	14.94	239.01	
28	40200	14.95	229.01	
28	38400	14.95	218.75	
28	48800	14.94	278.37	
28	45700	15.02	257.92	
42	52200	14.95	297.37	270.82
42	42800	15.10	239.00	
42	48400	14.94	276.09	

RESISTENCIA A LA COMPRESIÓN
a/c=0.60
CON DIFERENTES DOSIFICACIONES DE FIBRA





RESISTENCIA A LA COMPRESIÓN
a/c=0.65
CON DIFERENTES DOSIFICACIONES DE FIBRA
GRAFICO 8.22
RESISTENCIA (Kg/cm^2)
350.00
300.00
250.00
200.00
150.00
100.00
50.00
0.00
0
5
10
15
20
25
30
35
40
45
DIAS (Días)
FIBRA: INSONEX
LONGITUD: 40 mm
Patrón
30 F
40 F
50 F


RESISTENCIA A LA COMPRESIÓN
a/c=0.70
CON DIFERENTES DOSIFICACIONES DE FIBRA
GRAFICO 8.23
300.00
250.00
200.00
150.00
100.00
50.00
0.00
RESISTENCIA (Kg/cm2)
0
5
10
15
20
25
30
35
40
45
EDAD (Días)
FIBRA: INSONEX
LONGITUD: 40 mm
Patrón
30 F
40 F
50 F

8.2.2.2 ENSAYO DE RESISTENCIA A LA TRACCIÓN POR COMPRESIÓN DIAMETRAL (kg/cm²)

- RELACIÓN AGUA CEMENTO 0.60
  - **DOSIFICACIÓN DE LA FIBRA 35 Kg/m³ DE CONCRETO**

N° Días	Carga (Kg)	Diámetro (cm)	Luz (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
28	22000	14.80	30.00	31.56	30.36
28	20400	14.85	30.00	29.15	

  - **DOSIFICACIÓN DE LA FIBRA 45 Kg/m³ DE CONCRETO**

N° Días	Carga (Kg)	Diámetro (cm)	Luz (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
28	22600	14.90	30.10	32.08	32.11
28	22800	15.00	30.10	32.15	

  - **DOSIFICACIÓN DE LA FIBRA 55 Kg/m³ DE CONCRETO**

N° Días	Carga (Kg)	Diámetro (cm)	Luz (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
28	24050	15.10	30.20	33.57	34.67
28	25450	15.00	30.20	35.77	

- RELACIÓN AGUA CEMENTO 0.65
  - **DOSIFICACIÓN DE LA FIBRA 35 Kg/m³ DE CONCRETO**

N° Días	Carga (Kg)	Diámetro (cm)	Luz (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
28	18350	14.80	30.00	26.31	25.41
28	17100	14.80	30.00	24.52	

- DOSIFICACIÓN DE LA FIBRA 45 Kg/m³ DE CONCRETO

N° Días	Carga (Kg)	Diámetro (cm)	Luz (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
28	20100	14.80	30.10	28.72	27.51
28	18750	14.83	30.60	26.30	

- DOSIFICACIÓN DE LA FIBRA 55 Kg/m³ DE CONCRETO

N° Días	Carga (Kg)	Diámetro (cm)	Luz (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
28	17850	15.05	30.40	24.85	24.46
28	17300	15.10	30.30	24.07	

> **RELACIÓN AGUA CEMENTO 0.70**

- DOSIFICACIÓN DE LA FIBRA 35 Kg/m³ DE CONCRETO

N° Días	Carga (Kg)	Diámetro (cm)	Luz (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
28	17250	14.90	30.00	24.57	25.45
28	18550	14.90	30.10	26.33	

- DOSIFICACIÓN DE LA FIBRA 45 Kg/m³ DE CONCRETO

N° Días	Carga (Kg)	Diámetro (cm)	Luz (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
28	20100	14.80	30.10	28.72	27.51
28	18750	14.83	30.60	26.30	

- **DOSIFICACIÓN DE LA FIBRA 55 Kg/m³ DE CONCRETO**

N° Días	Carga (Kg)	Diámetro (cm)	Luz (cm)	Resistencia (Kg/cm²)	Promedio (Kg/cm²)
28	18000	15.05	30.00	25.38	25.70
28	18450	15.05	30.00	26.01	

8.2.2.3 ENSAYO DE MÓDULO DE ELASTICIDAD ESTÁTICO (10^5 kg/cm²)

- **RELACIÓN AGUA CEMENTO 0.60**

Dosificación Fibra kg/m³	Modulo Elástico Estático x100000 kg/cm²
35	2.8970
45	2.9670
55	3.0900

- **RELACIÓN AGUA CEMENTO 0.65**

Dosificación Fibra kg/m³	Modulo Elástico Estático x100000 kg/cm²
35	2.7980
45	2.8340
55	2.9110

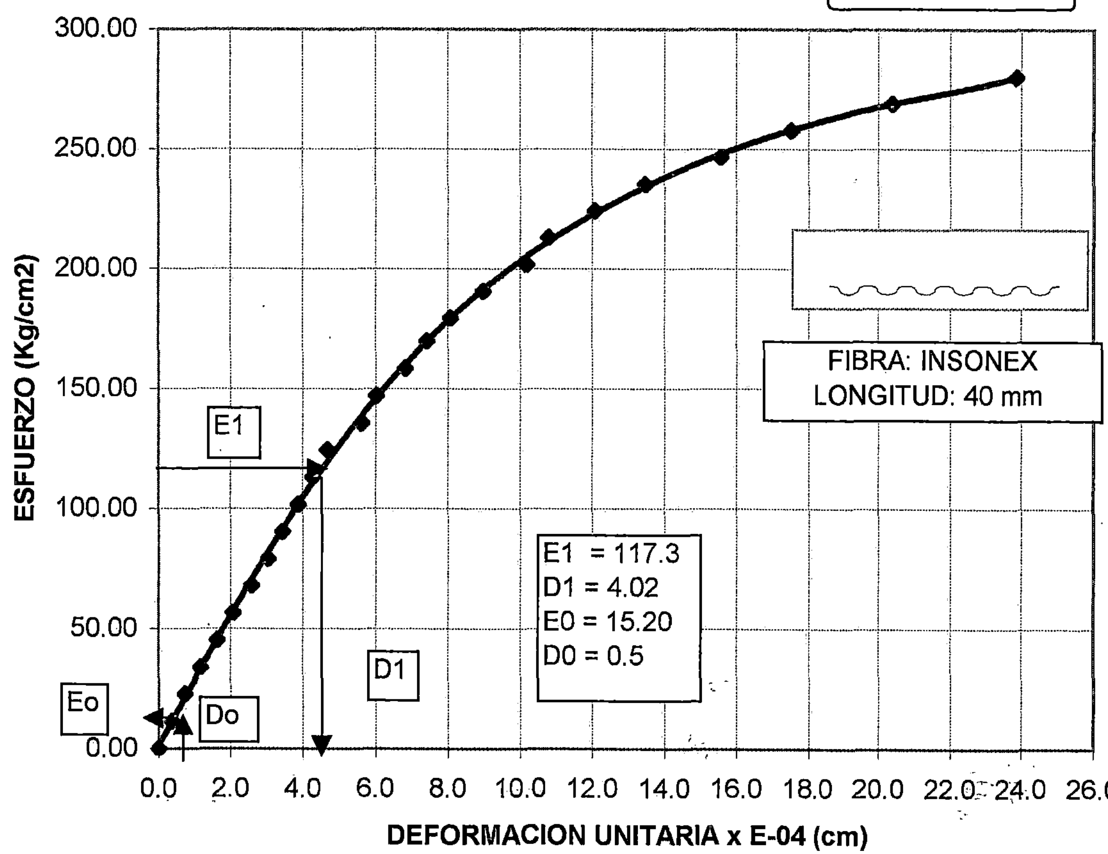
- **RELACIÓN AGUA CEMENTO 0.70**

Dosificación Fibra kg/m³	Modulo Elástico Estático x100000 kg/cm²
35	2.6040
45	2.6570
55	2.7560

Esfuerzo	Def.Unit
0.00	0.00
11.32	0.38
22.64	0.73
33.95	1.18
45.27	1.63
56.59	2.08
67.91	2.58
79.22	3.03
90.54	3.43
101.86	3.88
113.18	4.28
124.49	4.68
135.81	5.63
147.13	6.03
158.45	6.83
169.76	7.43
179.37	8.08
190.58	8.98
201.79	10.18
213.00	10.78
224.22	12.08
235.43	13.48
246.64	15.58
257.85	17.53
269.06	20.38
280.27	23.88

MÓDULO ELÁSTICO ESTÁTICO RELACIÓN a/c= 0.60
DOSIFICACIÓN DE FIBRA: 35 Kg/m³ DE CONCRETO

GRAFICO 8.24

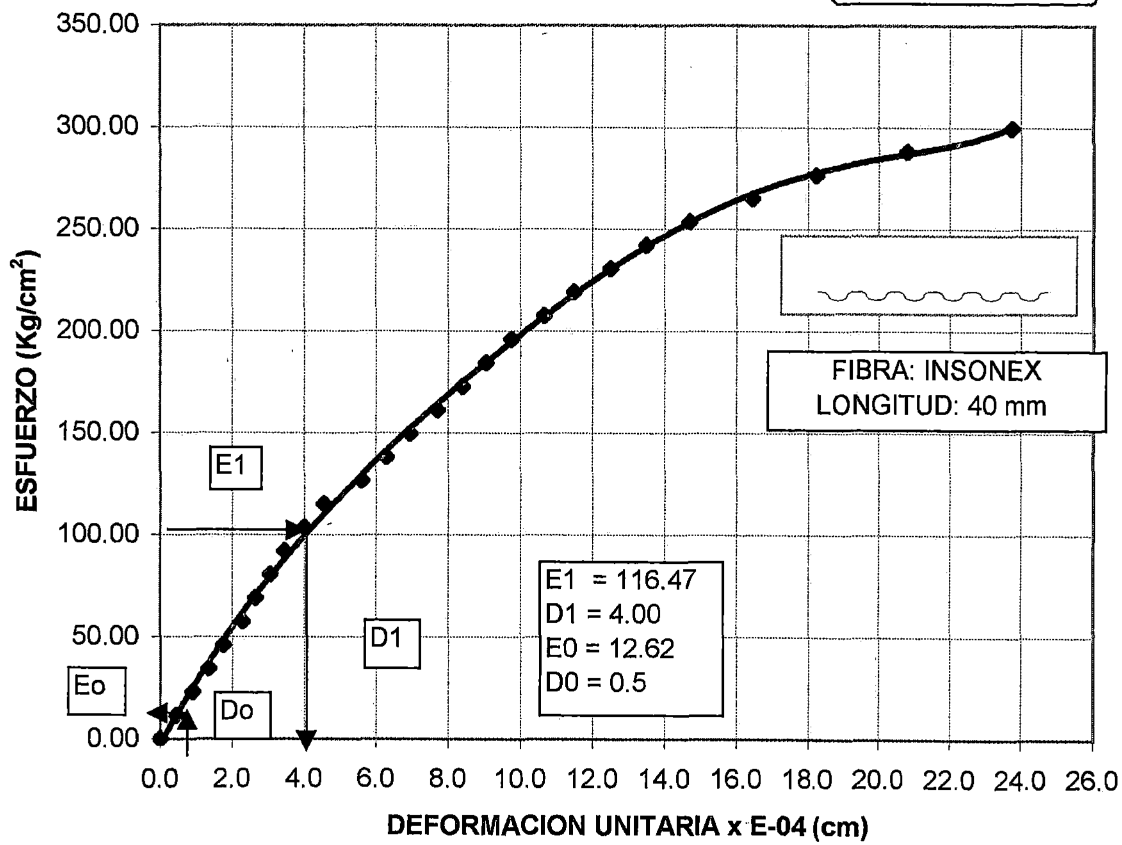


Esfuerzo	Def.Unit
0.00	0.00
11.53	0.45
23.06	0.90
34.60	1.35
46.13	1.75
57.66	2.30
69.19	2.65
80.72	3.05
92.26	3.45
103.79	4.00
115.32	4.55
126.85	5.60
138.38	6.30
149.91	6.95
161.45	7.70
172.98	8.40
184.51	9.05
196.04	9.75
207.57	10.65
219.11	11.50
230.64	12.50
242.17	13.50
253.70	14.70
265.23	16.45
276.77	18.25
288.30	20.80
299.83	23.75

MÓDULO ELÁSTICO ESTÁTICO RELACIÓN a/c=0.60

DOSIFICACIÓN DE FIBRA: 45 Kg/m³ DE CONCRETO

GRAFICO 8.25

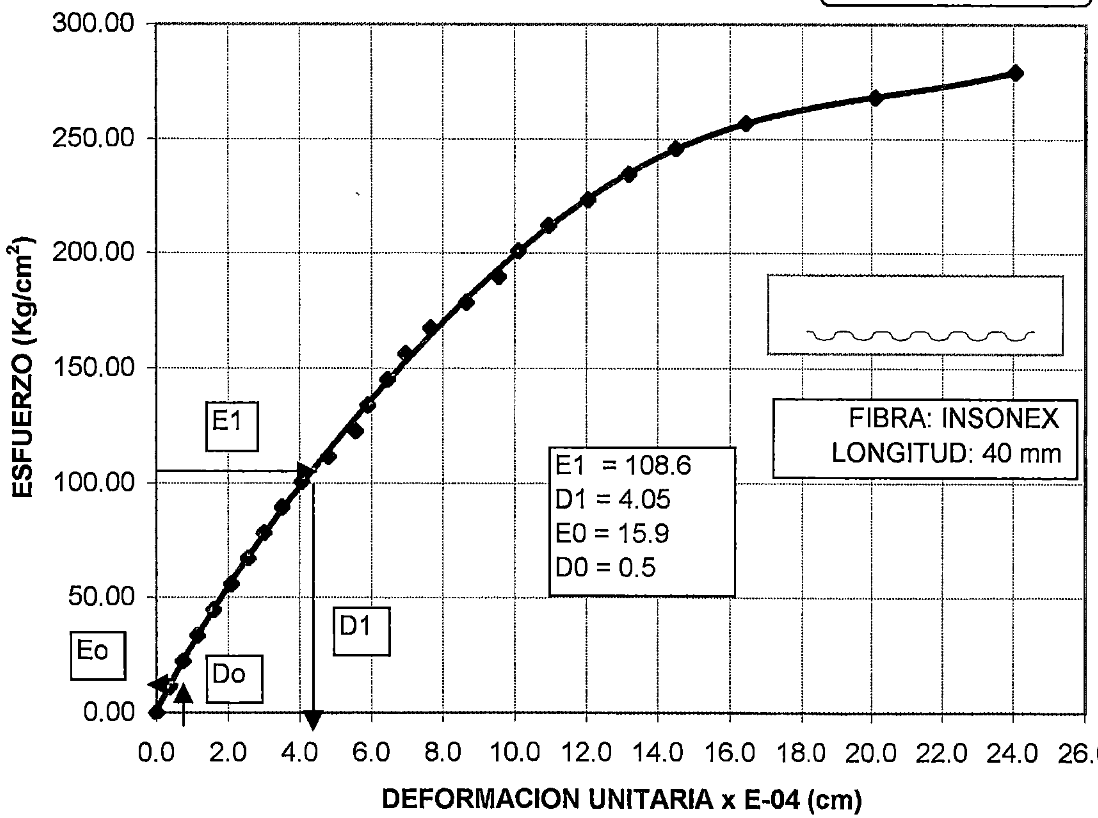


Tesis: "Estudio del comportamiento del concreto de mediana a baja resistencia, con la incorporación de Fibras de Acero y CPT I Andino"

Esfuerzo	Def.Unit
0.00	0.00
11.17	0.35
22.34	0.75
33.50	1.15
44.67	1.60
55.84	2.10
67.01	2.55
78.18	3.00
89.35	3.50
100.51	4.05
111.68	4.80
122.85	5.55
134.02	5.90
145.19	6.45
156.36	6.95
167.52	7.65
178.69	8.65
189.86	9.55
201.03	10.10
212.20	10.95
223.37	12.05
234.53	13.20
245.70	14.50
256.87	16.45
268.04	20.10
279.21	24.05

MÓDULO ELÁSTICO ESTÁTICO RELACIÓN a/c=0.60
DOSIFICACIÓN DE FIBRA: 55 Kg/m^3 DE CONCRETO

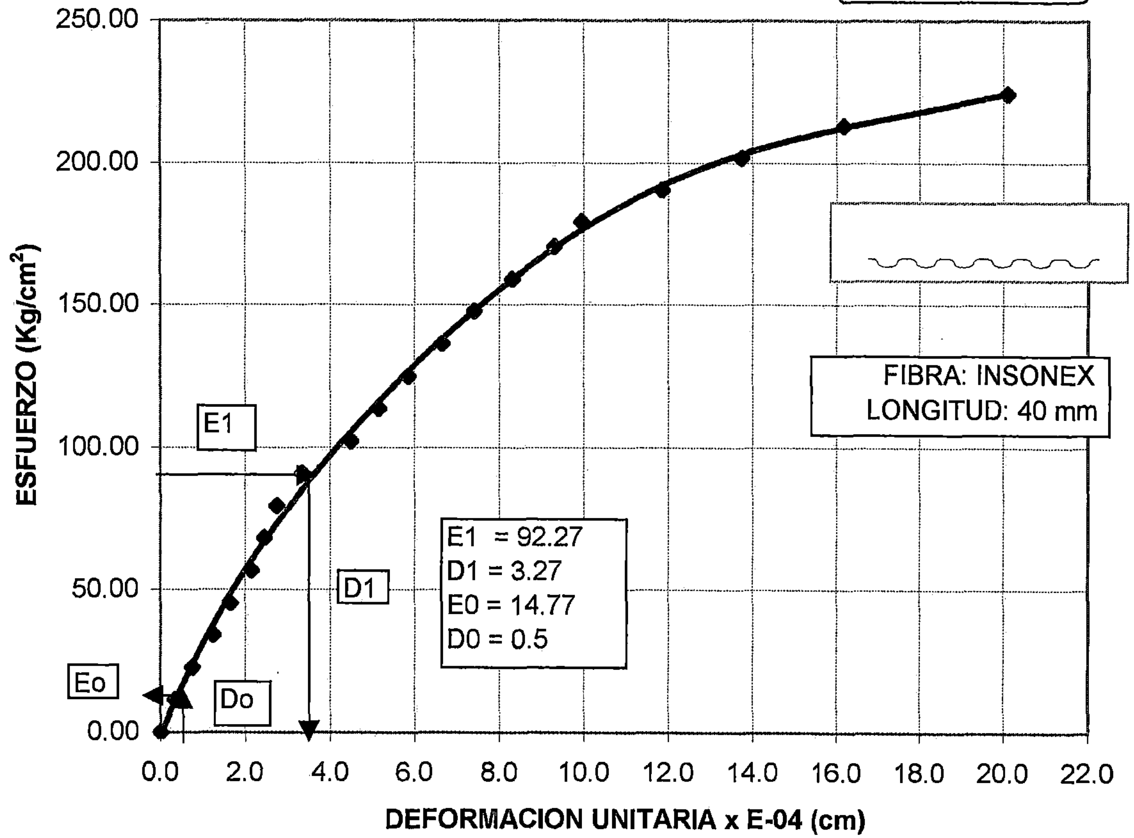
GRAFICO 8.26



MÓDULO ELÁSTICO ESTÁTICO RELACIÓN a/c= 0.65
DOSIFICACIÓN DE FIBRA: 35 Kg/m^3 DE CONCRETO

Esfuerzo	Def.Unit
0.00	0.00
11.36	0.35
22.73	0.75
34.09	1.25
45.45	1.65
56.82	2.15
68.18	2.45
79.54	2.75
90.90	3.35
102.27	4.50
113.63	5.15
124.99	5.85
136.36	6.65
147.72	7.40
159.08	8.30
170.45	9.30
179.37	9.95
190.58	11.85
201.79	13.75
213.00	16.20
224.22	20.10

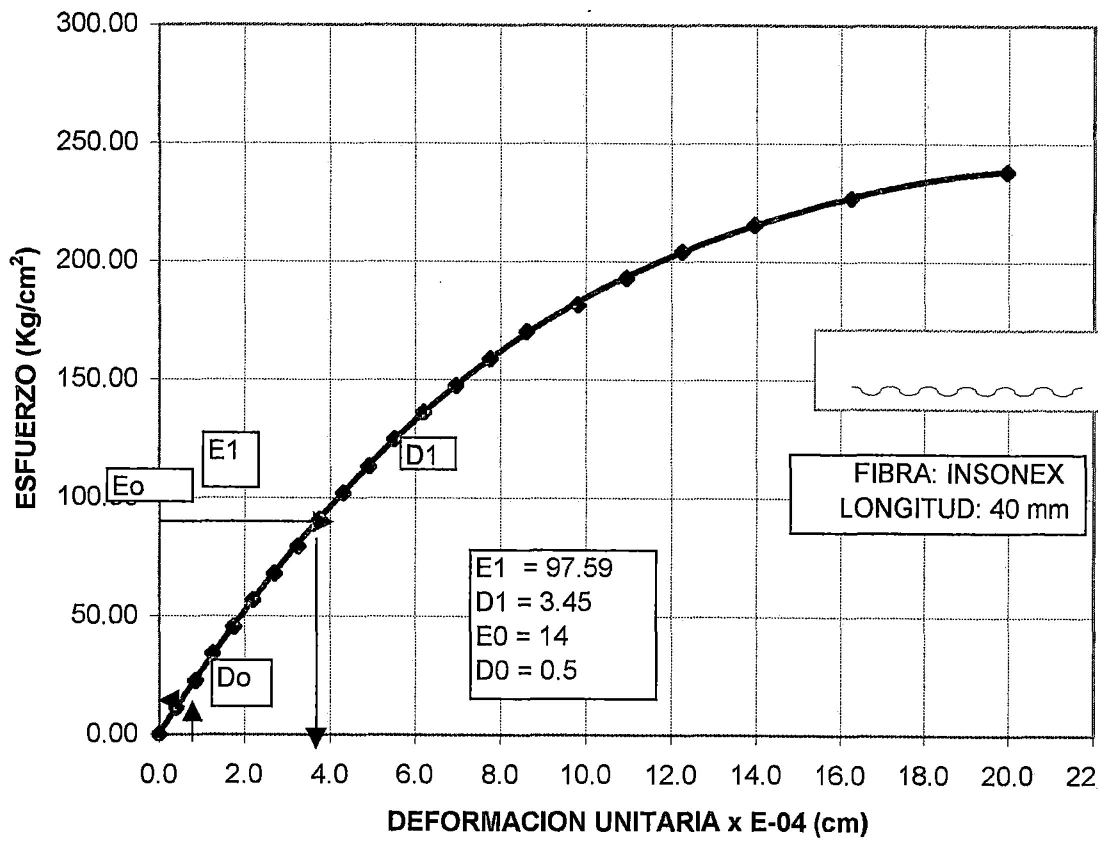
GRAFICO 8.27



Esfuerzo	Def.Unit
0.00	0.00
11.35	0.40
22.70	0.85
34.04	1.25
45.39	1.75
56.74	2.20
68.09	2.70
79.44	3.25
90.78	3.75
102.13	4.30
113.48	4.90
124.83	5.50
136.17	6.20
147.52	6.95
158.87	7.75
170.22	8.60
181.57	9.80
192.91	10.95
204.26	12.25
215.61	13.95
226.96	16.25
238.31	19.95

MÓDULO ELÁSTICO ESTÁTICO RELACIÓN a/c=0.65
DOSIFICACIÓN DE FIBRA: 45 Kg/m³ DE CONCRETO

GRAFICO 8.28

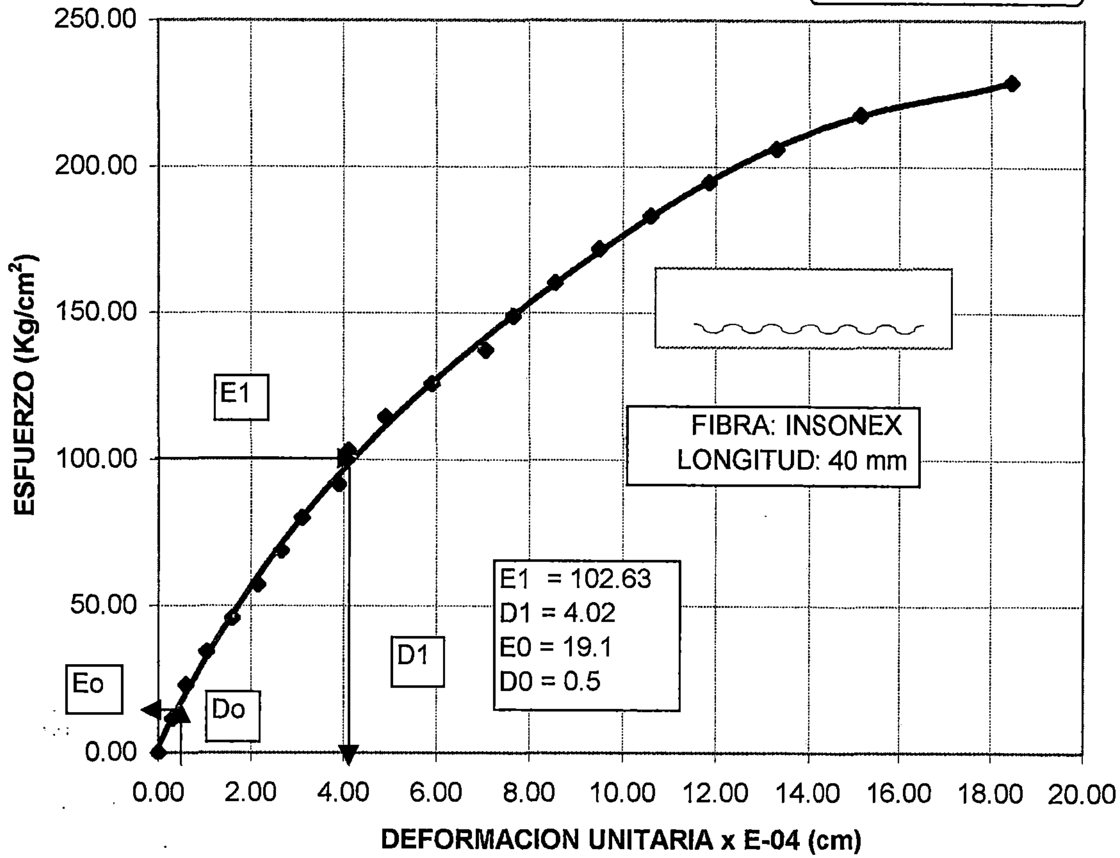


Tesis: "Estudio del comportamiento del concreto de mediana a baja resistencia, con la incorporación de Fibras de Acero y CPT I Andino"

MÓDULO ELÁSTICO ESTÁTICO RELACIÓN a/c=0.65
DOSIFICACIÓN DE FIBRA: 55 Kg/m^3 DE CONCRETO

Esfuerzo	Def.Unit
0.00	0.00
11.45	0.30
22.91	0.60
34.36	1.05
45.82	1.60
57.27	2.15
68.73	2.65
80.18	3.10
91.64	3.90
103.09	4.10
114.55	4.90
126.00	5.90
137.46	7.05
148.91	7.65
160.37	8.55
171.82	9.50
183.28	10.60
194.73	11.85
206.18	13.30
217.64	15.15
229.09	18.45

GRAFICO 8.29

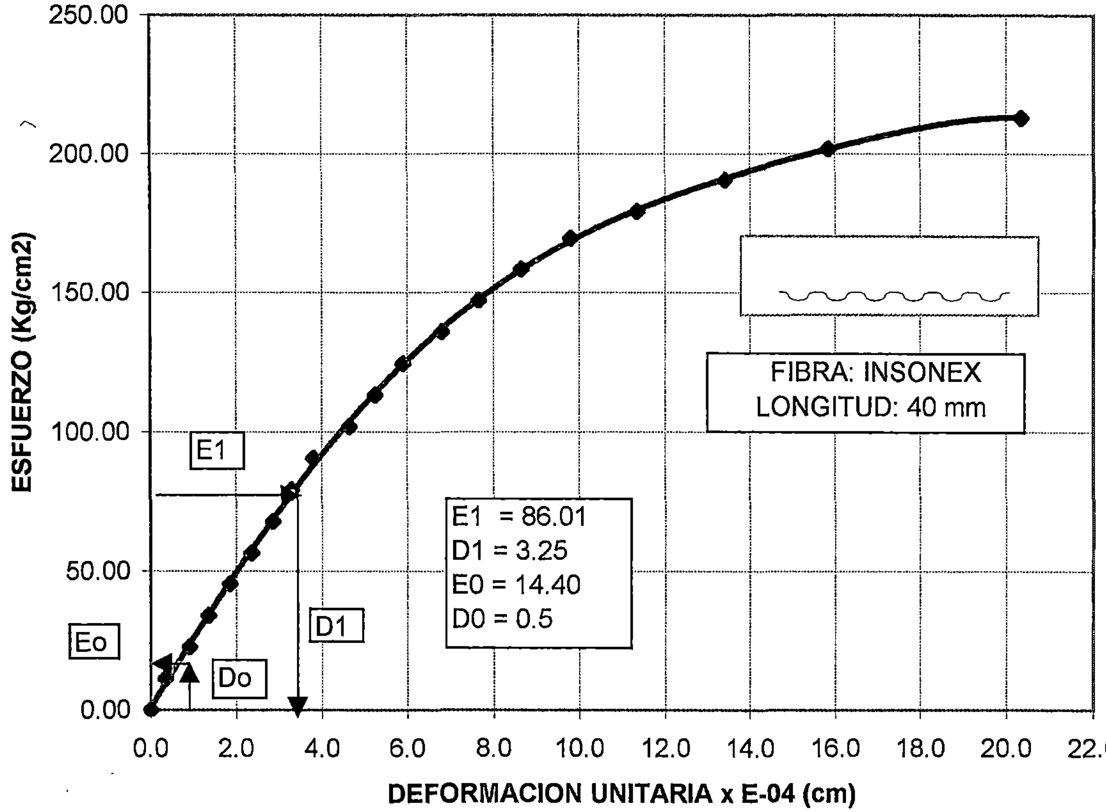


Tesis: "Estudio del comportamiento del concreto de mediana a baja resistencia, con la incorporación de Fibras de Acero y CPT I Andino"

MÓDULO ELÁSTICO ESTÁTICO RELACIÓN a/c= 0.70
DOSIFICACIÓN DE FIBRA: 35 Kg/m^3 DE CONCRETO

GRAFICO 8.30

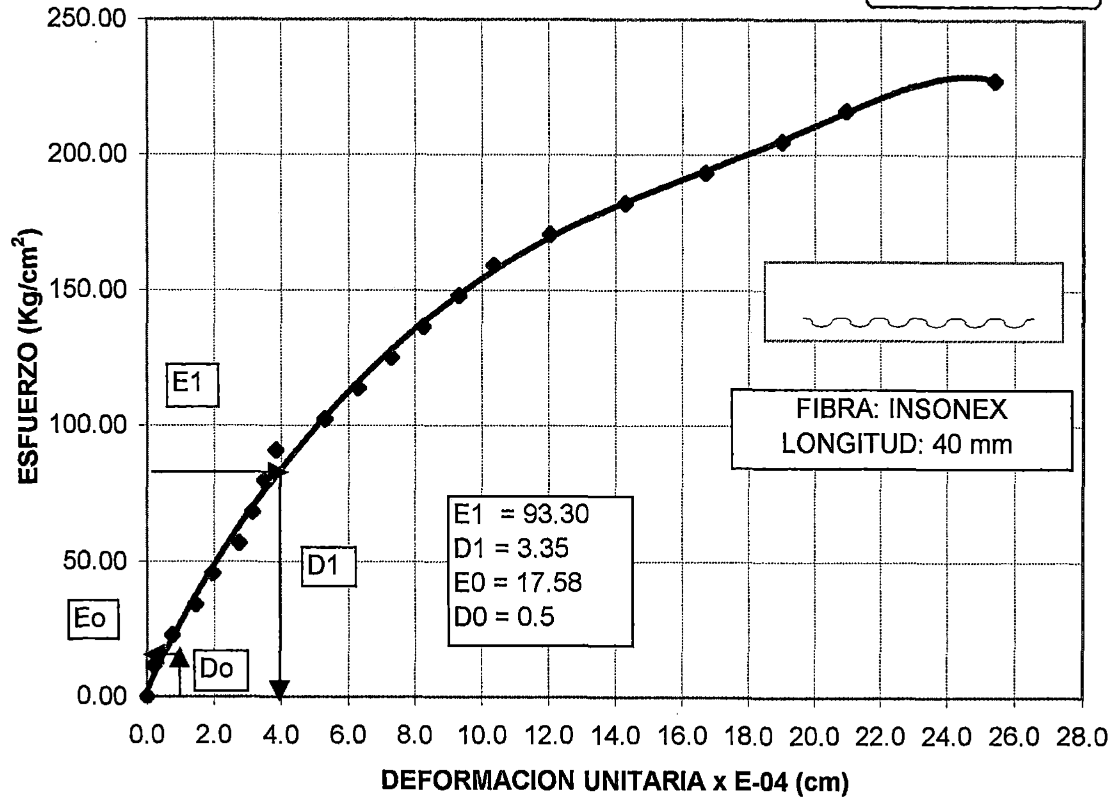
Esfuerzo	Def.Unit
0.00	0.00
11.32	0.35
22.64	0.90
33.95	1.35
45.27	1.85
56.59	2.35
67.91	2.85
79.22	3.30
90.54	3.80
101.86	4.65
113.18	5.25
124.49	5.90
135.81	6.80
147.13	7.65
158.45	8.65
169.76	9.80
179.37	11.35
190.58	13.40
201.79	15.85
213.00	20.35



Esfuerzo	Def.Unit
0.00	0.00
11.38	0.20
22.76	0.75
34.13	1.45
45.51	1.95
56.89	2.75
68.27	3.15
79.65	3.50
91.03	3.85
102.40	5.30
113.78	6.30
125.16	7.30
136.54	8.25
147.92	9.30
159.30	10.35
170.67	12.05
182.05	14.30
193.43	16.70
204.81	19.00
216.19	20.95
227.57	25.40

MODULO ELASTICO ESTATICO RELACION a/c=0.70
DOSIFICACION DE FIBRA: 45Kg/m^3 DE CONCRETO

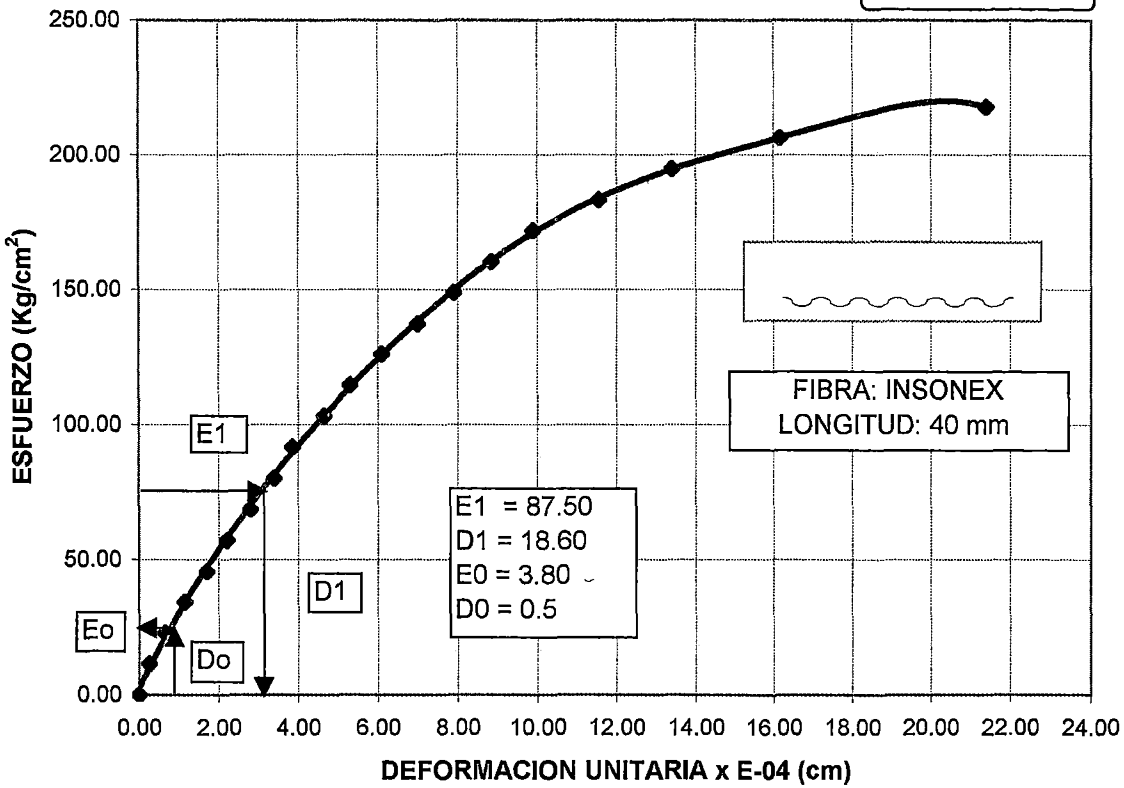
GRAFICO 8.31



MÓDULO ELÁSTICO ESTÁTICO RELACIÓN a/c=0.70
DOSIFICACIÓN DE FIBRA: 55 Kg/m^3 DE CONCRETO

GRAFICO 8.32

Esfuerzo	Def.Unit
0.00	0.00
11.45	0.25
22.91	0.65
34.36	1.15
45.82	1.70
57.27	2.20
68.73	2.80
80.18	3.40
91.64	3.85
103.09	4.65
114.55	5.30
126.00	6.10
137.46	7.00
148.91	7.90
160.37	8.85
171.82	9.90
183.28	11.55
194.73	13.40
206.18	16.15
217.64	21.40



Tesis: "Estudio del comportamiento del concreto de mediana a baja resistencia, con la incorporación de Fibras de Acero y CPT I Andino"

8.2.2.4 ENSAYO DE RESISTENCIA A LA FLEXION (kg/cm^2)

- **RELACIÓN AGUA CEMENTO 0.60**

  - **DOSIFICACIÓN DE LA FIBRA 35 Kg/m^3 DE CONCRETO**

N°Dias	Carga (Kg)	Base (cm)	Altura (cm)	Luz (cm)	Modulo (Kg/cm^2)	Promedio (Kg/cm^2)
28	2600	15.10	15.30	60.00	44.13	43.89
28	2420	15.20	15.00	60.00	42.46	
28	2570	15.00	15.10	60.00	45.09	

  - **DOSIFICACIÓN DE LA FIBRA 45 Kg/m^3 DE CONCRETO**

N°Dias	Carga (Kg)	Base (cm)	Altura (cm)	Luz (cm)	Modulo (Kg/cm^2)	Promedio (Kg/cm^2)
28	2600	15.10	15.30	60.00	44.13	43.89
28	2420	15.20	15.00	60.00	42.46	
28	2570	15.00	15.10	60.00	45.09	

  - **DOSIFICACIÓN DE LA FIBRA 55 Kg/m^3 DE CONCRETO**

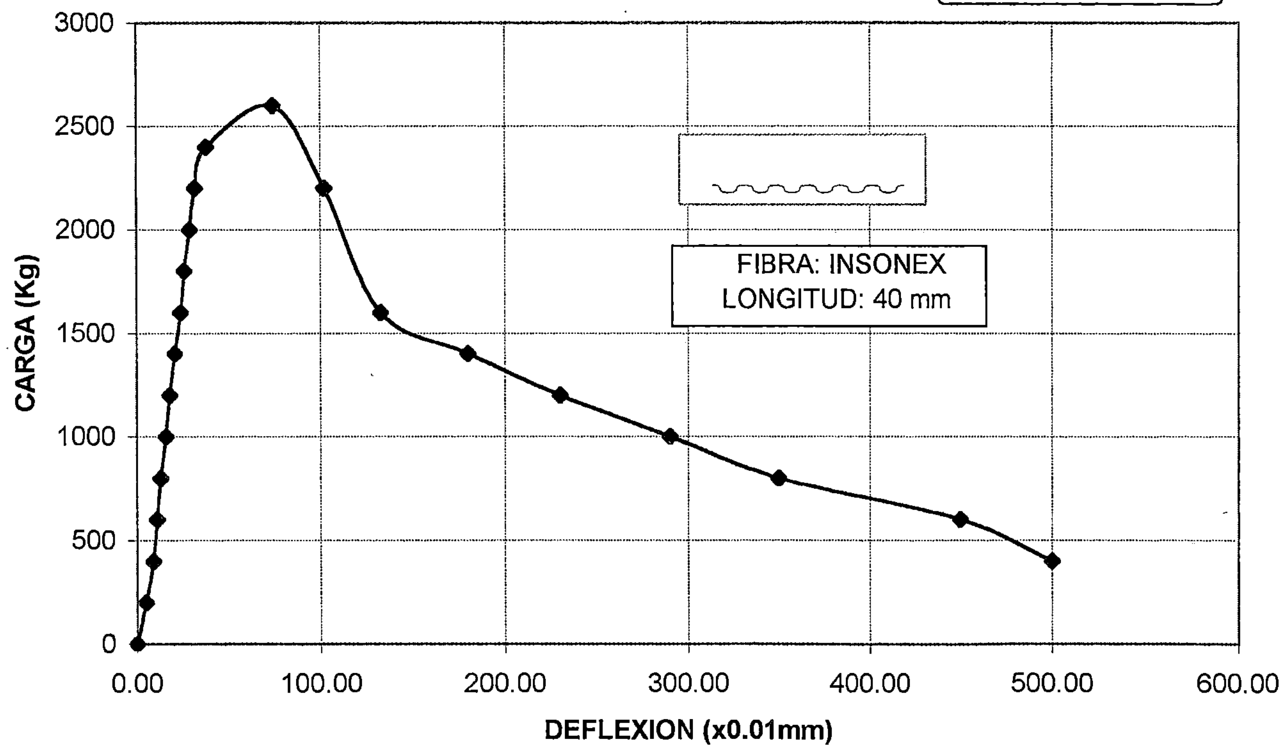
N°Dias	Carga (Kg)	Base (cm)	Altura (cm)	Luz (cm)	Modulo (Kg/cm^2)	Promedio (Kg/cm^2)
28	2960	15.30	15.20	60.00	50.24	48.88
28	2710	15.10	15.30	60.00	46.00	
28	2970	15.30	15.20	60.00	50.41	

Carga (Kg)	Deflexión (x0.01mm)
0	0.00
200	5.00
400	9.00
600	11.00
800	13.00
1000	16.00
1200	18.00
1400	21.00
1600	24.00
1800	26.00
2000	29.00
2200	32.00
2400	38.00
2600	74.33
2200	102.00
1600	132.67
1400	180.00
1200	230.00
1000	290.00
800	350.00
600	450.00
400	500.00

Fluencia	**2600**

ENSAYO DE FLEXION RELACION a/c=0.60
DOSIFICACION DE LA FIBRA 35 Kg/m^3 DE CONCRETO

GRAFICO 8.34



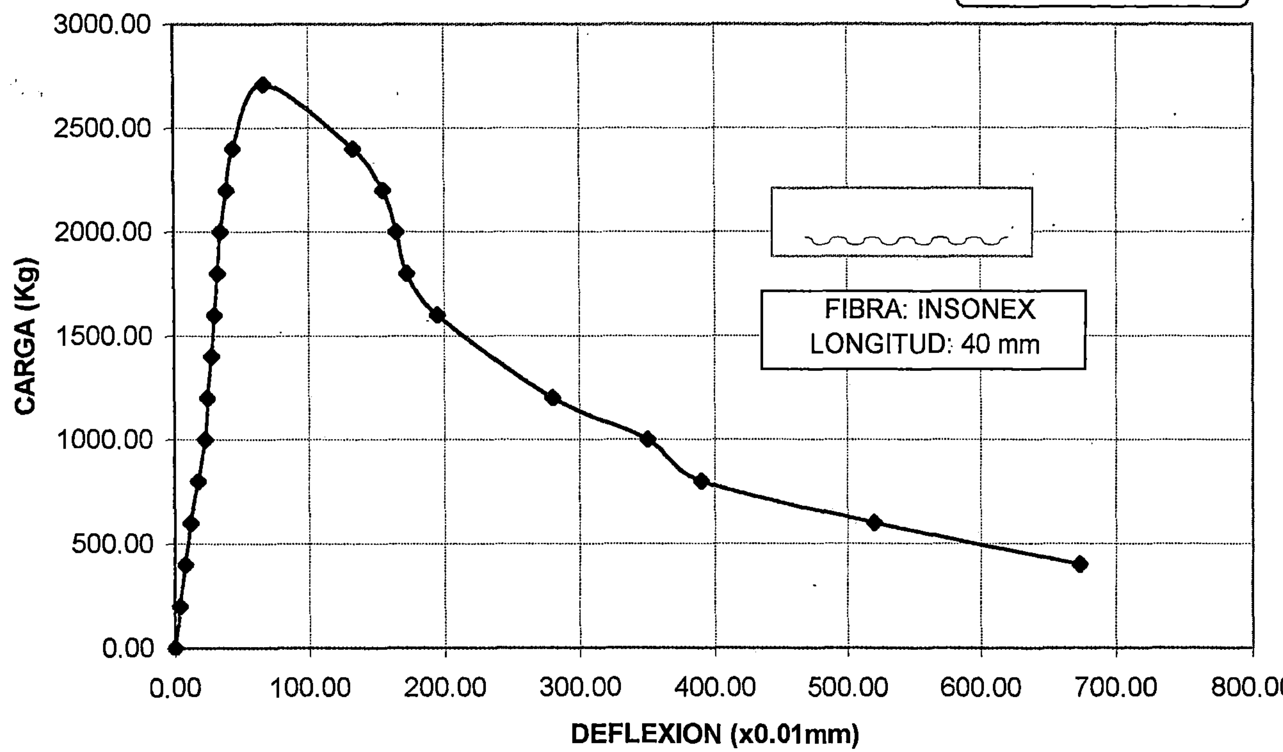
Carga (Kg)	Deflexión (x0.01mm)
0.00	0.00
200.00	4.00
400.00	8.00
600.00	12.00
800.00	18.00
1000.00	23.00
1200.00	25.00
1400.00	28.00
1600.00	30.00
1800.00	33.00
2000.00	35.00
2200.00	39.00
2400.00	44.33
2710.00	66.67
2400.00	133.33
2200.00	155.00
2000.00	165.00
1800.00	173.00
1600.00	195.00
1200.00	280.00
1000.00	350.00
800.00	390.00
600.00	520.00
400.00	673.33

Fluencia	2710

ENSAYO DE FLEXION RELACION a/c=0.60

DOSIFICACION DE LA FIBRA 45 Kg/m^3 DE CONCRETO

GRAFICO 8.35

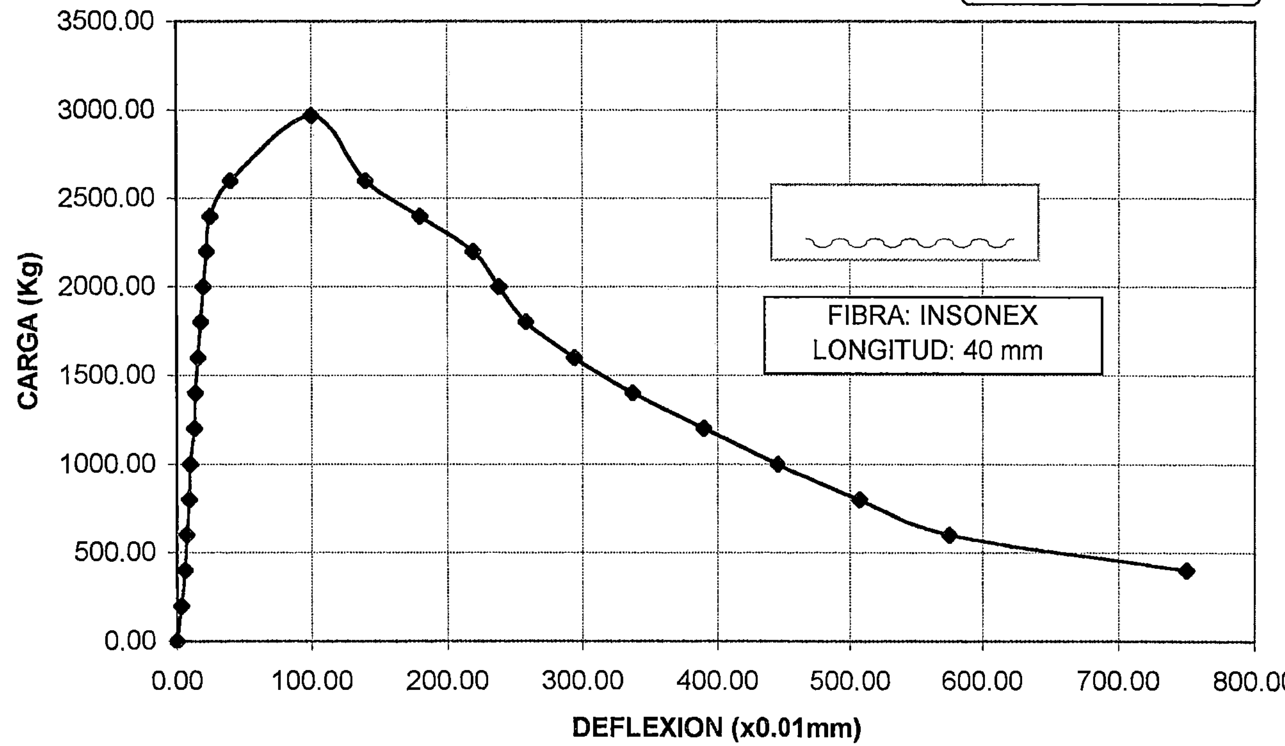


Carga (Kg)	Deflexión (x0.01mm)
0.00	0.00
200.00	3.00
400.00	6.00
600.00	7.00
800.00	9.00
1000.00	10.00
1200.00	13.00
1400.00	14.00
1600.00	16.00
1800.00	18.00
2000.00	20.00
2200.00	22.00
2400.00	25.00
2600.00	40.00
2970.00	100.00
2600.00	140.00
2400.00	180.00
2200.00	219.00
2000.00	238.00
1800.00	258.00
1600.00	294.00
1400.00	337.00
1200.00	390.00
1000.00	445.00
800.00	507.00
600.00	575.00
400.00	750.00

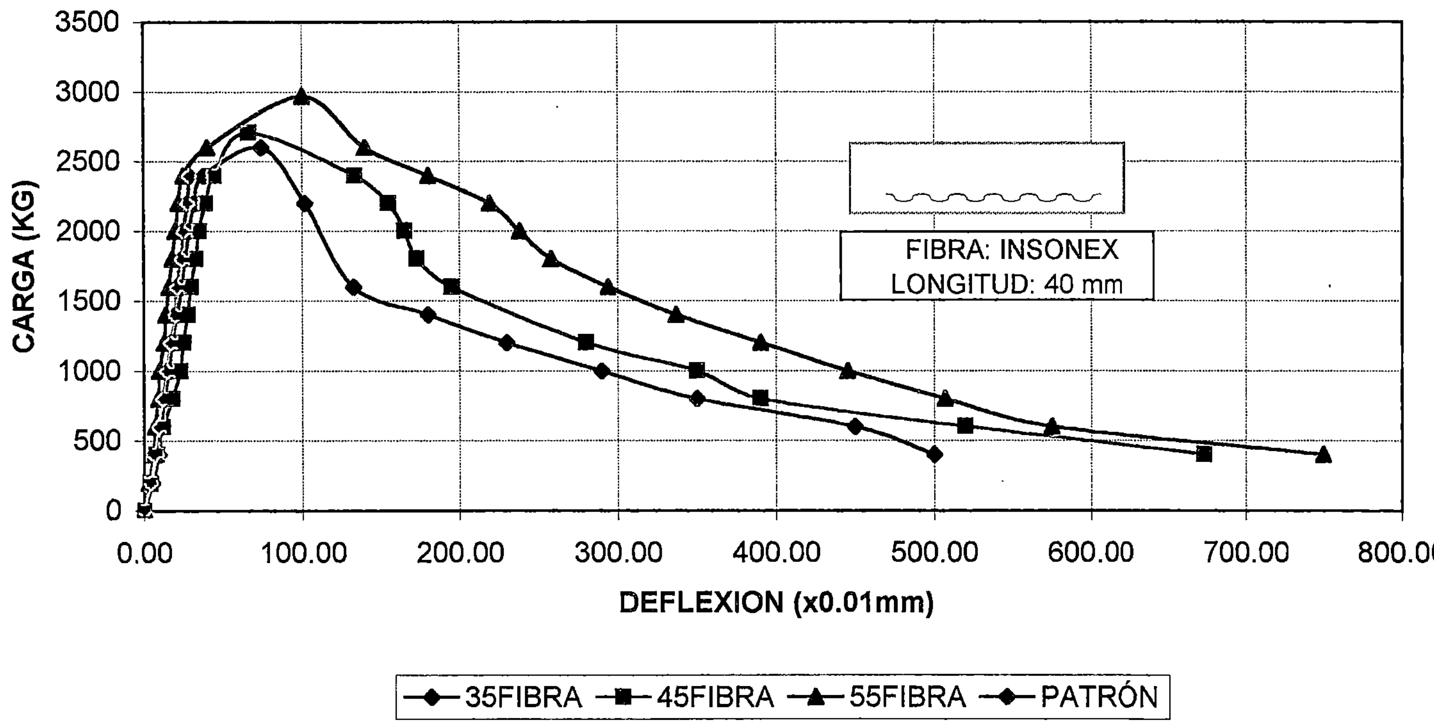
Fluencia	**2970**

ENSAYO DE FLEXION RELACION a/c=0.60
DOSIFICACION DE LA FIBRA 55 Kg/m^3 DE CONCRETO

GRAFICO 8.36



ENSAYO DE FLEXIÓN
GRÁFICA COMPARATIVA CON DIFERENTES DOSIFICACIONES DE FIBRA
RELACIÓN A/C:0.60



➢ **RELACIÓN AGUA CEMENTO 0.65**

- **DOSIFICACIÓN DE LA FIBRA 35 Kg/m³ DE CONCRETO**

N°Dias	Carga (Kg)	Base (cm)	Altura (cm)	Luz (cm)	Modulo (Kg/cm²)	Promedio (Kg/cm²)
28	2260	15.00	15.00	60.00	40.18	40.38
28	2300	15.30	15.00	60.00	40.09	
28	2330	15.20	15.00	60.00	40.88	

- **DOSIFICACIÓN DE LA FIBRA 45 Kg/m³ DE CONCRETO**

N°Dias	Carga (Kg)	Base (cm)	Altura (cm)	Luz (cm)	Modulo (Kg/cm²)	Promedio (Kg/cm²)
28	2320	15.00	15.00	60.00	41.24	41.11
28	2290	15.10	15.00	60.00	40.44	
28	2390	15.30	15.00	60.00	41.66	

- **DOSIFICACIÓN DE LA FIBRA 55 Kg/m³ DE CONCRETO**

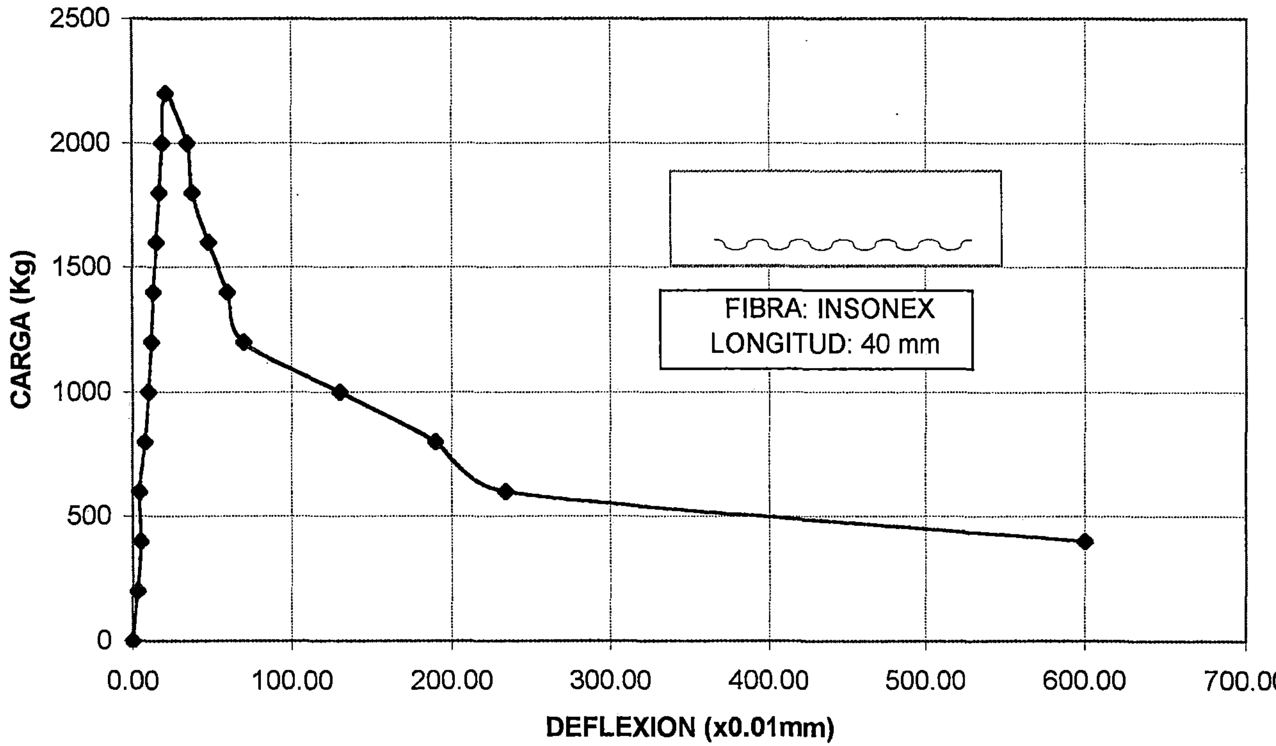
N°Dias	Carga (Kg)	Base (cm)	Altura (cm)	Luz (cm)	Modulo (Kg/cm²)	Promedio (Kg/cm²)
28	2530	15.00	15.00	60.00	44.98	43.85
28	2480	15.10	15.20	60.00	42.65	
28	2570	15.00	15.30	60.00	43.91	

Carga (Kg)	Deflexión (x0.01mm)
0	0.00
200	3.00
400	5.00
600	4.50
800	8.00
1000	10.00
1200	12.00
1400	13.50
1600	15.00
1800	17.00
2000	19.00
2200	21.00
2000	35.00
1800	38.00
1600	48.00
1400	60.00
1200	70.00
1000	130.00
800	190.00
600	234.00
400	600.00

Fluencia	**2330**

RESISTENCIA A LA FLEXION RELACION a/c=0.65
DOSIFICACION DE LA FIBRA 35 Kg/m³ DE CONCRETO

GRAFICO 8.37

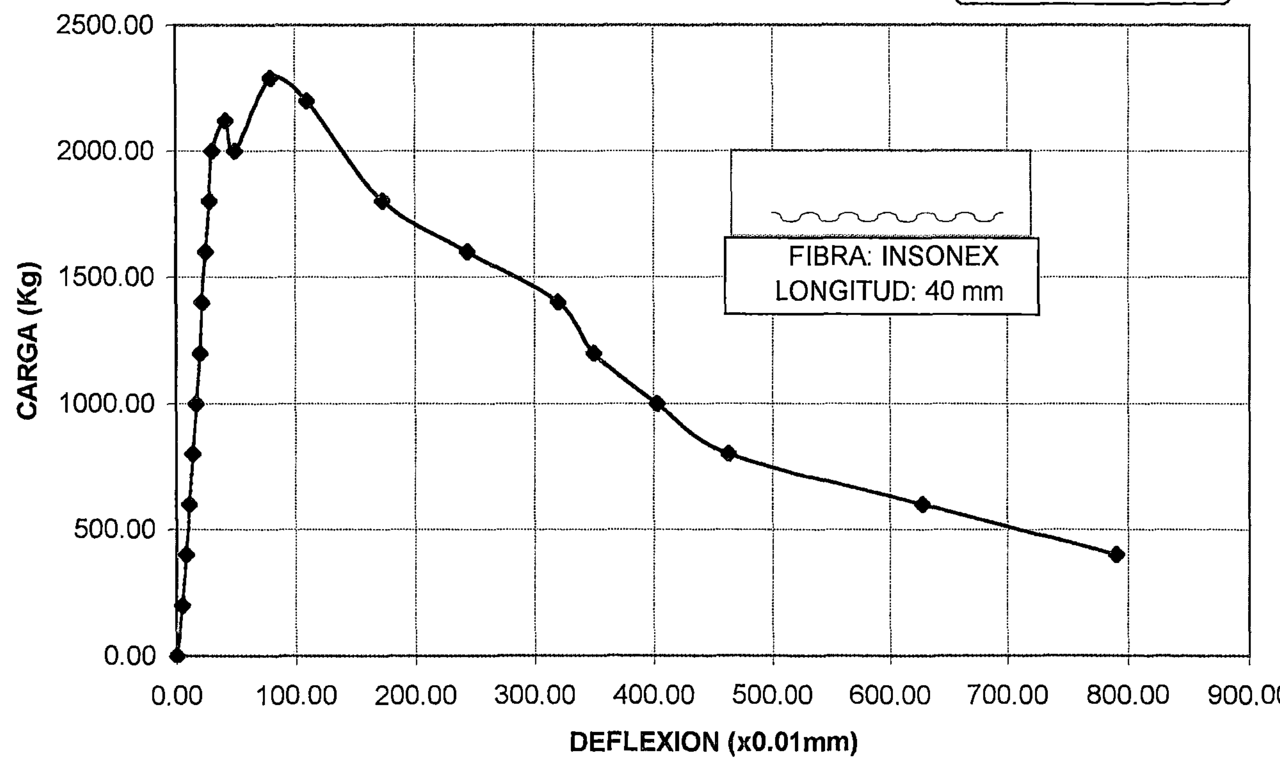


Carga (Kg)	Deflexión (x0.01mm)
0.00	0.00
200.00	5.00
400.00	8.00
600.00	11.00
800.00	14.00
1000.00	17.00
1200.00	20.00
1400.00	22.00
1600.00	25.00
1800.00	28.00
2000.00	31.00
2120.00	42.00
2000.00	50.00
2290.00	80.00
2200.00	110.00
1800.00	173.00
1600.00	244.00
1400.00	320.00
1200.00	350.00
1000.00	403.00
800.00	463.00
600.00	628.00
400.00	790.00

Fluencia	2290

RESISTENCIA A LA FLEXION RELACION a/c=0.65
DOSIFICACION DE LA FIBRA 45 Kg/m3 DE CONCRETO

GRAFICO 8.38

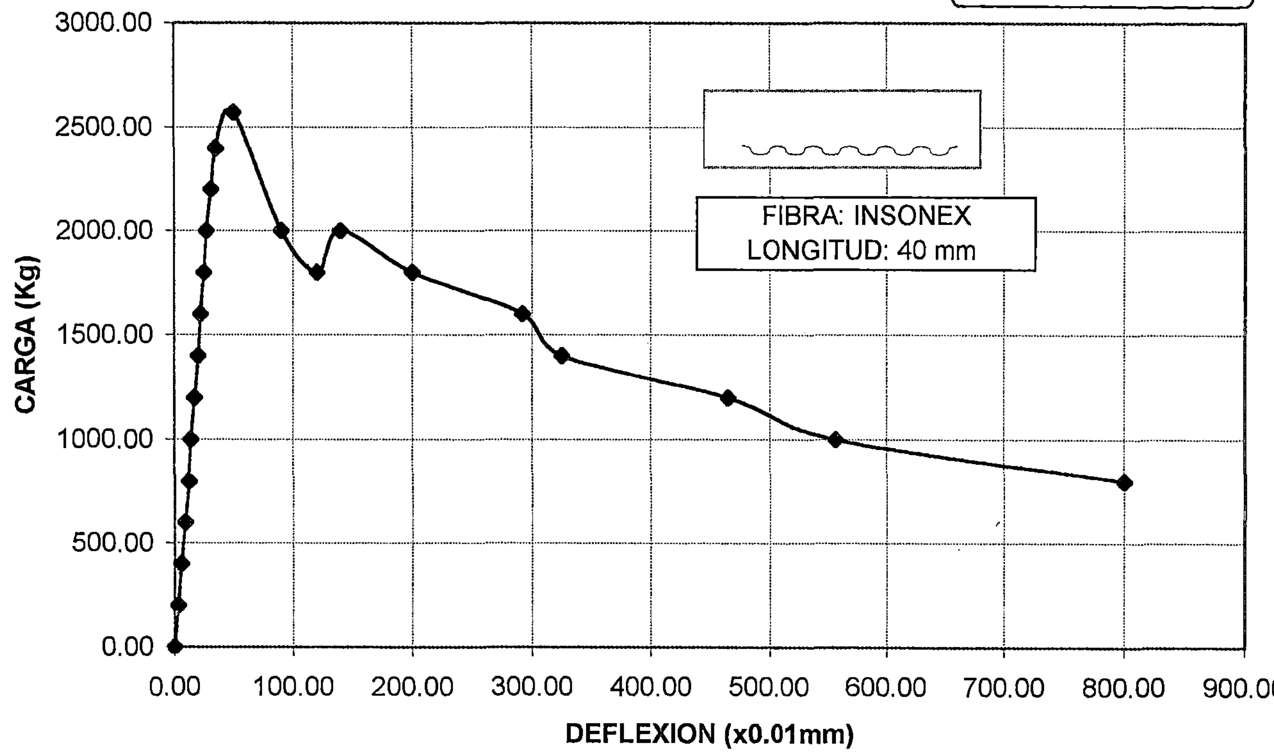


Carga (Kg)	Deflexión (x0.01mm)
0.00	0.00
200.00	3.00
400.00	6.00
600.00	9.00
800.00	12.00
1000.00	14.00
1200.00	17.00
1400.00	20.00
1600.00	22.00
1800.00	25.00
2000.00	27.00
2200.00	31.00
2400.00	35.00
2570.00	50.00
2000.00	90.00
1800.00	120.00
2000.00	140.00
1800.00	200.00
1600.00	292.00
1400.00	325.00
1200.00	465.00
1000.00	557.00
800.00	800.00

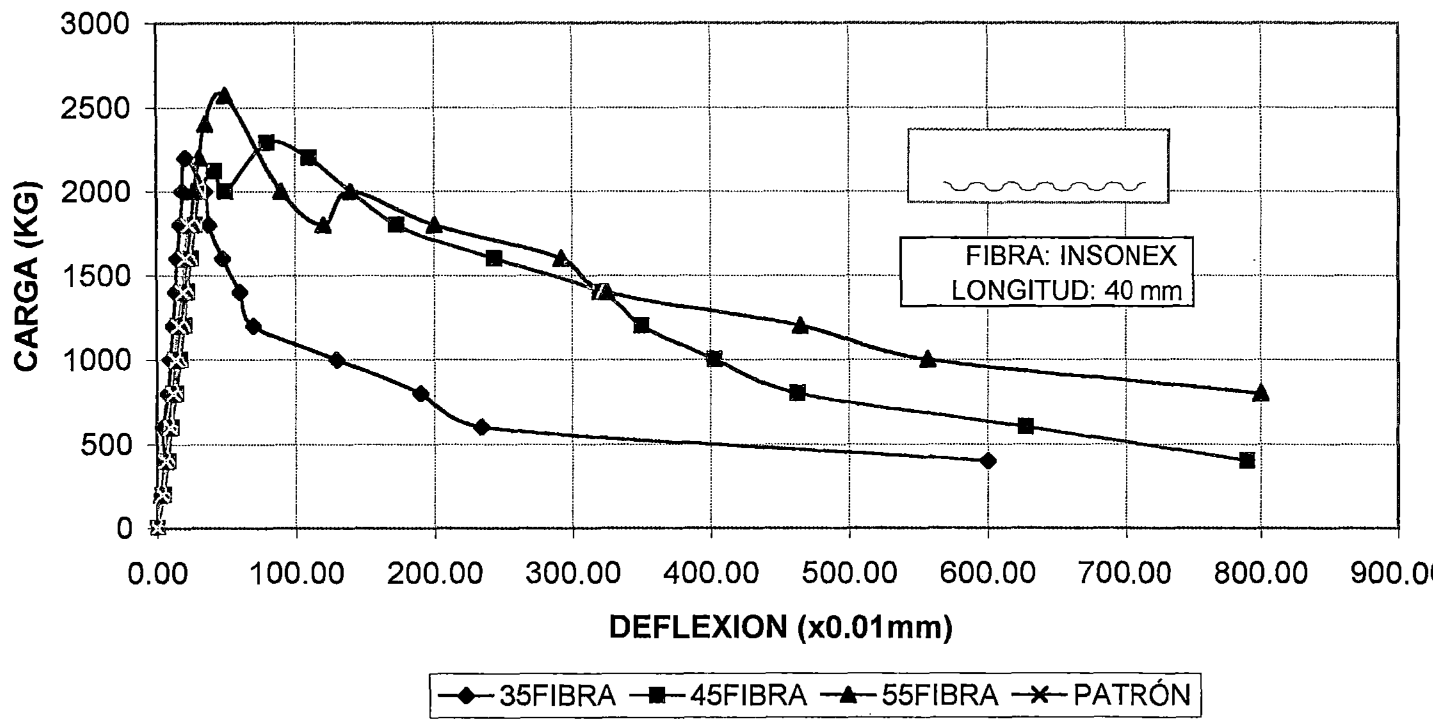
Fluencia	2570

RESISTENCIA A LA FLEXION RELACION a/c=0.65
DOSIFICACION DE LA FIBRA 55 Kg/m^3 DE CONCRETO

GRAFICO 8.39



ENSAYO DE FLEXIÓN
GRÁFICA COMPARATIVA CON DIFERENTES DOSIFICACIONES DE FIBRA
RELACIÓN A/C:0.65



➢ RELACIÓN AGUA CEMENTO 0.70

o DOSIFICACIÓN DE LA FIBRA 35 Kg/m³ DE CONCRETO

N°Dias	Carga (Kg)	Base (cm)	Altura (cm)	Luz (cm)	Modulo (Kg/cm²)	Promedio (Kg/cm²)
28	2240	15.20	15.30	60.00	37.77	36.63
28	2100	15.50	15.40	60.00	34.28	
28	2200	15.30	15.10	60.00	37.84	

o DOSIFICACIÓN DE LA FIBRA 45 Kg/m³ DE CONCRETO

N°Dias	Carga (Kg)	Base (cm)	Altura (cm)	Luz (cm)	Modulo (Kg/cm²)	Promedio (Kg/cm²)
28	2140	15.00	15.20	60.00	37.05	37.62
28	2180	15.20	15.00	60.00	38.25	
28	2170	15.00	15.20	60.00	37.57	

o DOSIFICACIÓN DE LA FIBRA 55 Kg/m³ DE CONCRETO

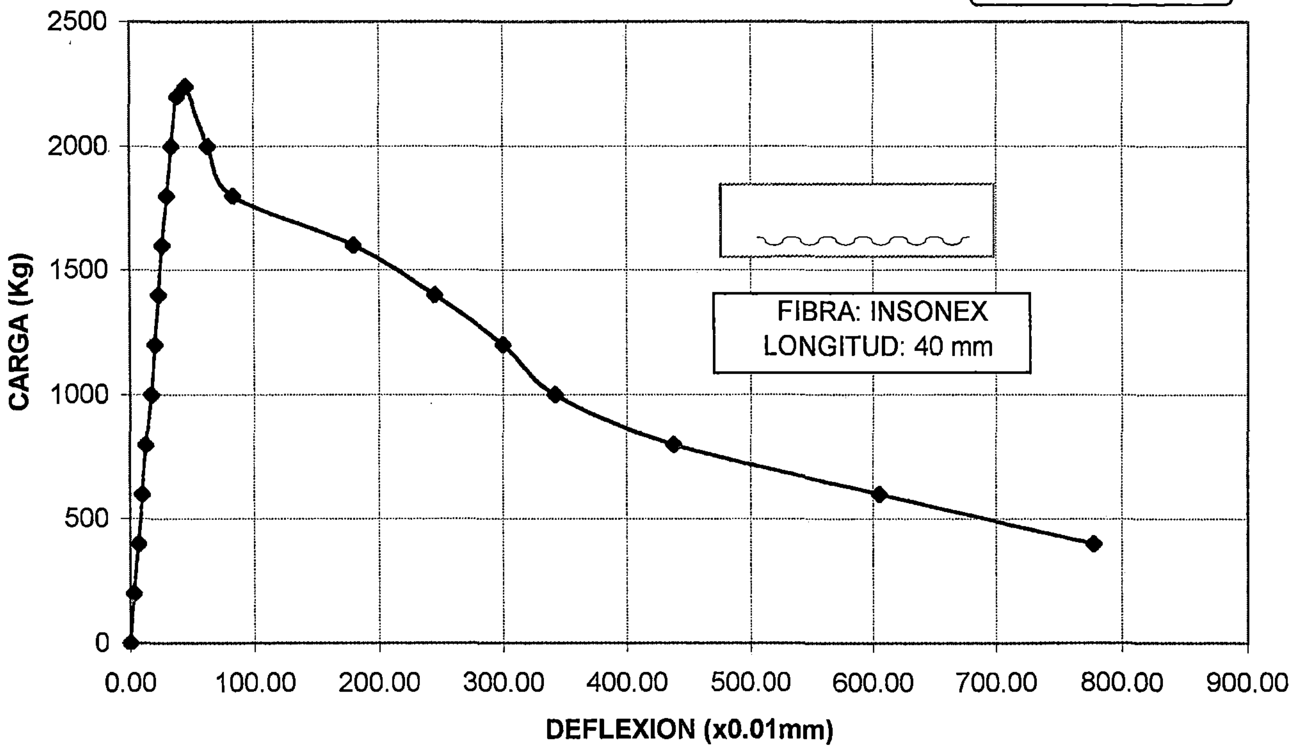
N°Dias	Carga (Kg)	Base (cm)	Altura (cm)	Luz (cm)	Modulo (Kg/cm2)	Promedio (Kg/cm2)
28	2350	15.30	15.20	60.00	39.89	39.86
28	2240	15.10	15.20	60.00	38.52	
28	2410	15.20	15.20	60.00	41.18	

Carga (Kg)	Deflexión (x0.01mm)
0	0.00
200	3.00
400	7.00
600	10.00
800	13.00
1000	17.00
1200	20.00
1400	23.00
1600	26.00
1800	30.00
2000	34.00
2200	38.00
2240	45.00
2000	63.00
1800	83.00
1600	180.00
1400	245.00
1200	300.00
1000	342.00
800	438.00
600	605.00
400	778.00

Fluencia	**2240**

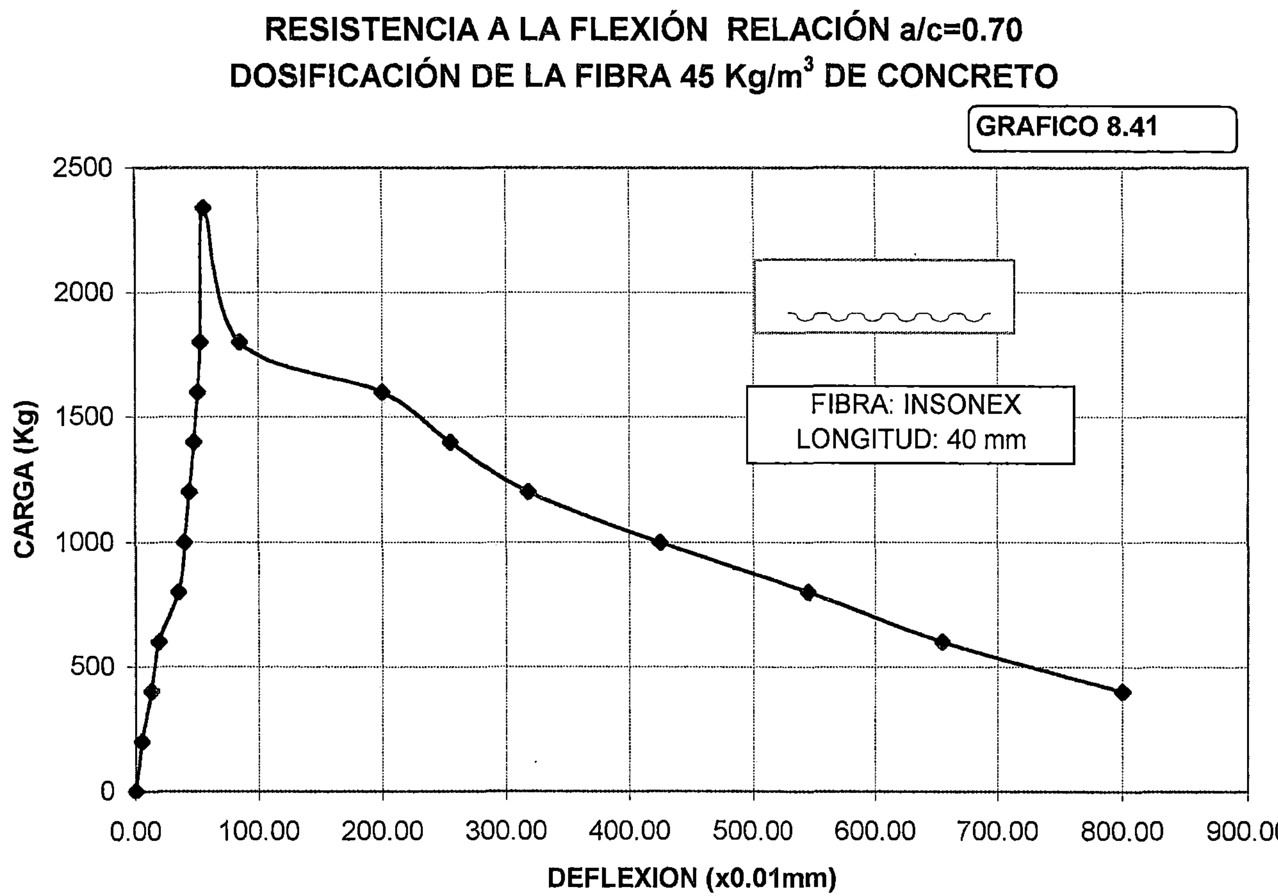
RESISTENCIA A LA FLEXIÓN RELACIÓN a/c=0.70
DOSIFICACIÓN DE LA FIBRA 35 Kg/m³ DE CONCRETO

GRAFICO 8.40



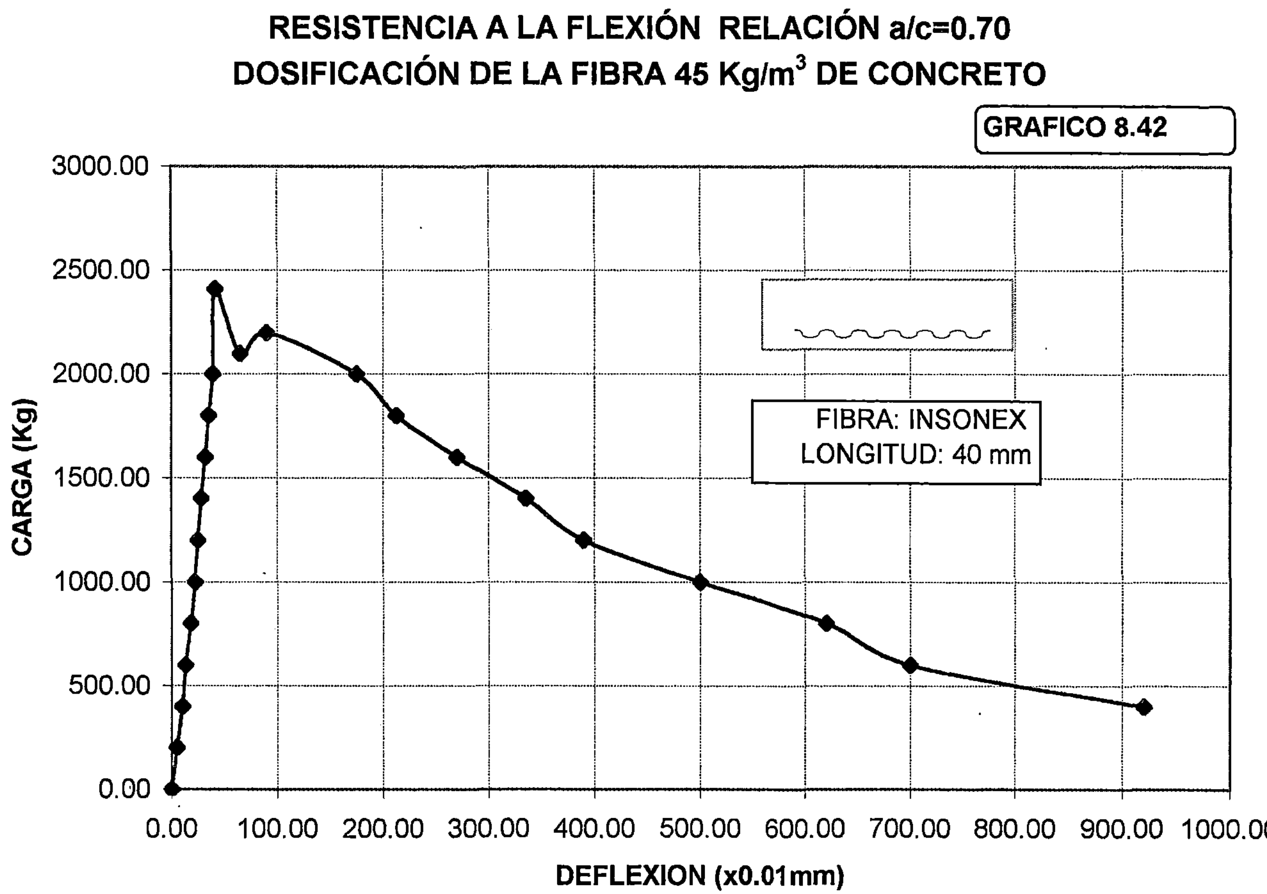
Carga (Kg)	Deflexión (x0.01mm)
0	0.00
200	5.00
400	13.00
600	19.00
800	35.00
1000	40.00
1200	44.00
1400	48.00
1600	51.00
1800	53.00
2340	56.00
1800	85.00
1600	200.00
1400	255.00
1200	318.00
1000	425.00
800	545.00
600	655.00
400	800.00

Fluencia	**2180**

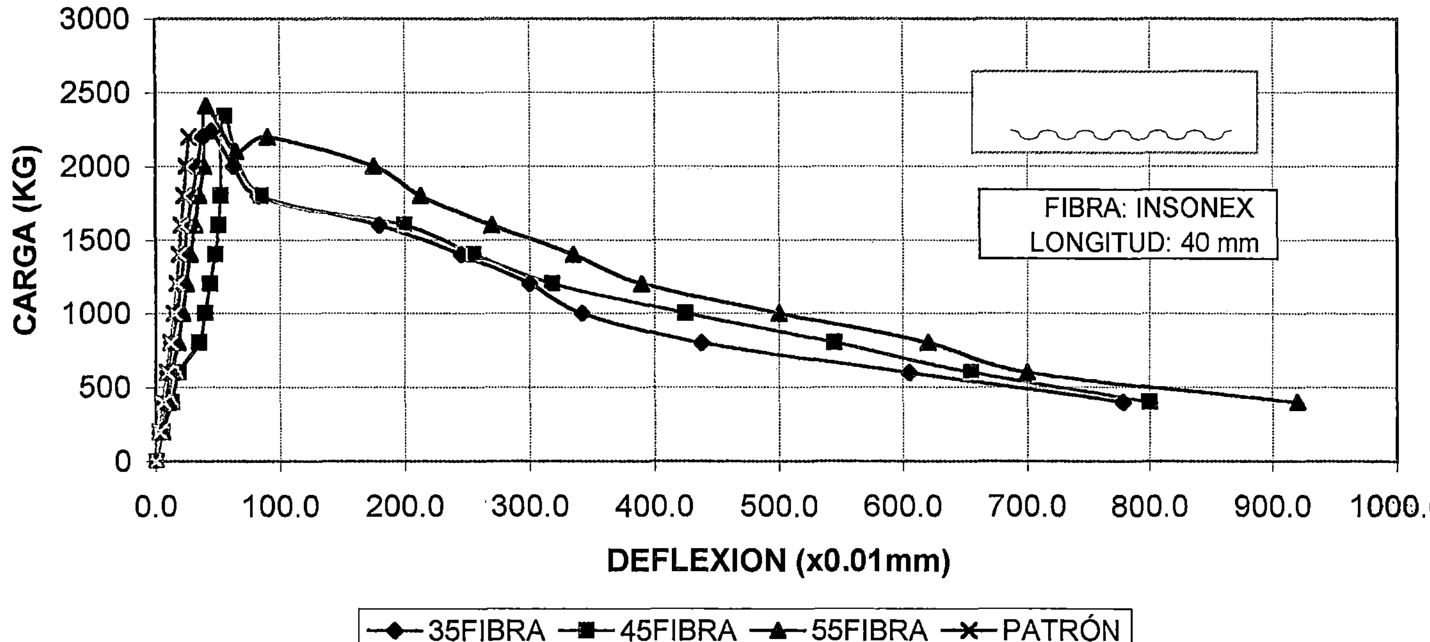


Carga (Kg)	Deflexión (x0.01mm)
0.00	0.00
200.00	5.00
400.00	10.00
600.00	13.00
800.00	18.00
1000.00	22.00
1200.00	25.00
1400.00	28.00
1600.00	32.00
1800.00	35.00
2000.00	39.00
2410.00	41.00
2100.00	65.00
2200.00	90.00
2000.00	175.00
1800.00	213.00
1600.00	270.00
1400.00	335.00
1200.00	390.00
1000.00	500.00
800.00	620.00
600.00	700.00
400.00	920.00

Fluencia	2410



ENSAYO DE FLEXIÓN
GRÁFICA COMPARATIVA CON DIFERENTES DOSIFICACIONES DE FIBRA
RELACIÓN A/C:0.70



8.2.2.5 ENSAYO DE RESISTENCIA AL IMPACTO (N° Golpes)

- **RELACION AGUA CEMENTO 0.60**

  - **DOSIFICACIÓN DE LA FIBRA 35 Kg/m³ DE CONCRETO**

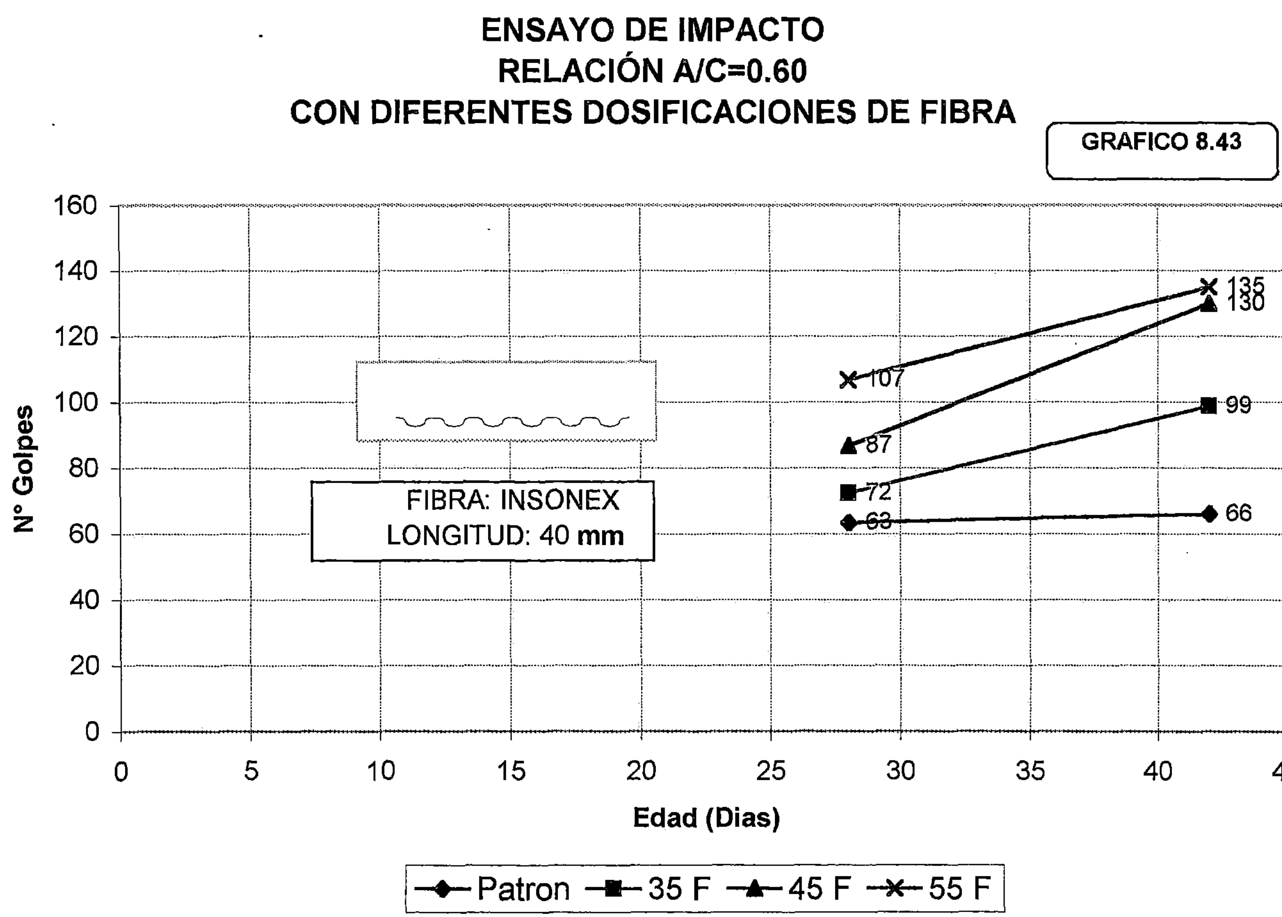
N° Días	Altura		Diámetro		N° Golpes	
	H	Prom	D	Prom	G	Prom
28	6.68	6.49	14.95	14.94	47	72
28	6.50		14.95		95	
28	6.30		14.92		75	
42	6.80	6.70	14.90	14.97	107	99
42	6.90		15.00		109	
42	6.40		15.00		80	

  - **DOSIFICACIÓN DE LA FIBRA 45 Kg/m³ DE CONCRETO**

N° Días	Altura		Diámetro		N° Golpes	
	H	Prom	D	Prom	G	Prom
28	6.20	6.46	15.10	15.03	75	87
28	6.58		15.00		125	
28	6.60		15.00		60	
42	6.90	6.70	14.90	14.97	150	130
42	6.50		15.00		90	
42	6.70		15.00		150	

  - **DOSIFICACIÓN DE LA FIBRA 55 Kg/m³ DE CONCRETO**

N° Días	Altura		Diámetro		N° Golpes	
	H	Prom	D	Prom	G	Prom
28	6.67	6.63	15.10	14.95	50	107
28	6.63		14.95		120	
28	6.6		14.80		150	
42	6.60	6.60	15.00	15.00	150	135
42	6.70		15.00		150	
42	6.50		15.00		105	



ENSAYO DE IMPACTO
RELACIÓN A/C=0.60
CON DIFERENTES DOSIFICACIONES DE FIBRA
GRAFICO 8.43
N° Golpes
Edad (Dias)
FIBRA: INSONEX
LONGITUD: 40 mm
Patron
35 F
45 F
55 F

➢ RELACIÓN AGUA CEMENTO 0.65

o DOSIFICACIÓN DE LA FIBRA 35 Kg/m³ DE CONCRETO

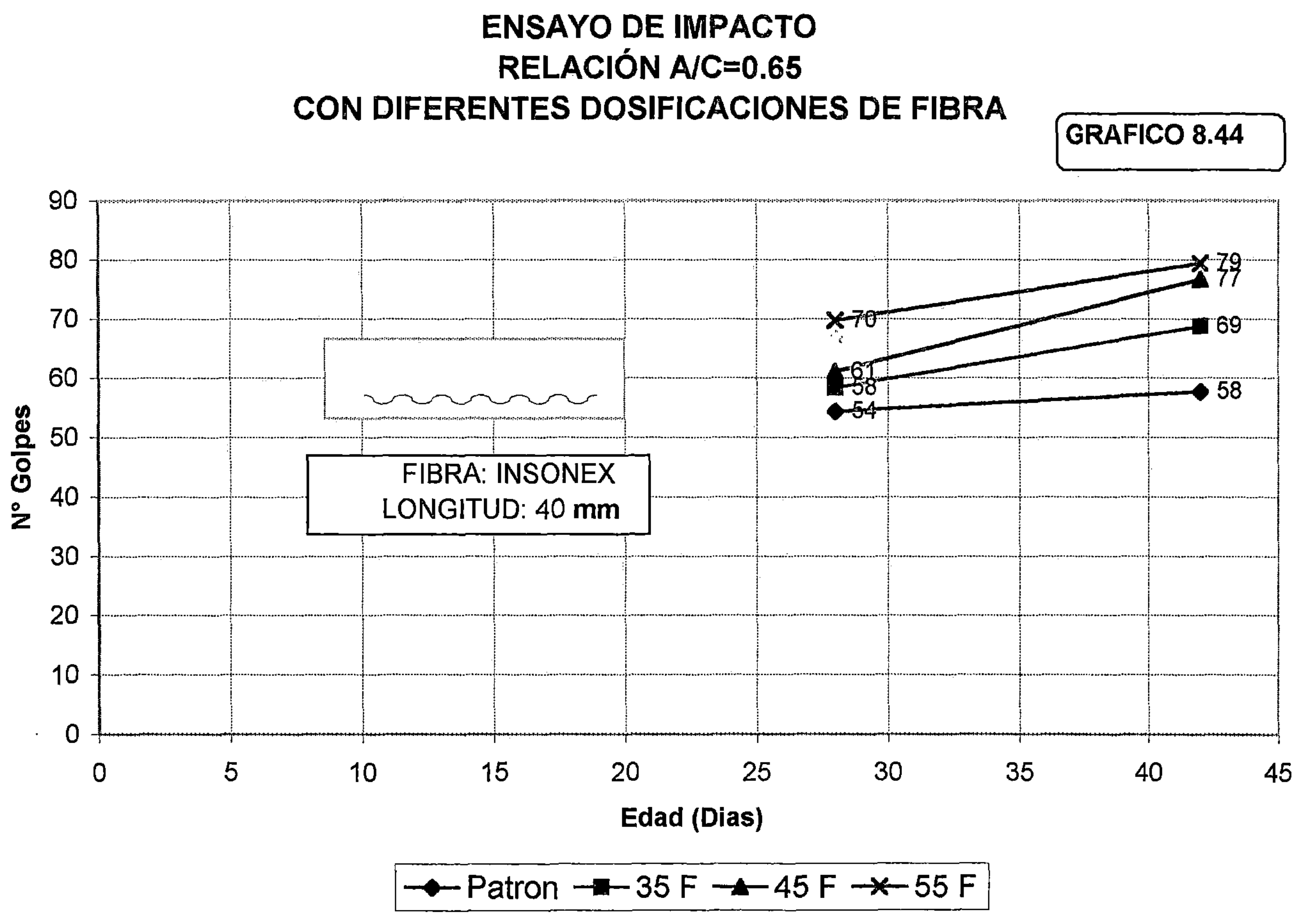
N° Días	Altura		Diámetro		N° Golpes	
	H	Prom	D	Prom	G	Prom
28	6.50	6.55	14.90	14.95	68	58
28	6.65		15.00		55	
28	6.50		14.95		52	
42	6.60	6.50	14.90	14.92	71	69
42	6.50		14.92		68	
42	6.40		14.95		67	

o DOSIFICACIÓN DE LA FIBRA 45 Kg/m³ DE CONCRETO

N° Días	Altura		Diámetro		N° Golpes	
	H	Prom	D	Prom	G	Prom
28	6.45	6.55	15.00	14.99	57	61
28	6.55		15.10		55	
28	6.65		14.88		71	
42	6.50	6.50	14.95	14.90	65	77
42	6.51		14.90		80	
42	6.50		14.85		85	

o DOSIFICACIÓN DE LA FIBRA 55 Kg/m³ DE CONCRETO

N° Días	Altura		Diámetro		N° Golpes	
	H	Prom	D	Prom	G	Prom
28	6.55	6.52	15.00	14.97	61	70
28	6.40		15.00		70	
28	6.60		14.90		78	
42	6.50	6.45	14.90	14.91	75	79
42	6.45		14.92		79	
42	6.40		14.92		84	



Tesis: "Estudio del comportamiento del concreto de mediana a baja resistencia, con la incorporación de Fibras de Acero y CPT I Andino"

➢ RELACIÓN AGUA CEMENTO 0.70

o DOSIFICACIÓN DE LA FIBRA 35 Kg/m^3 DE CONCRETO

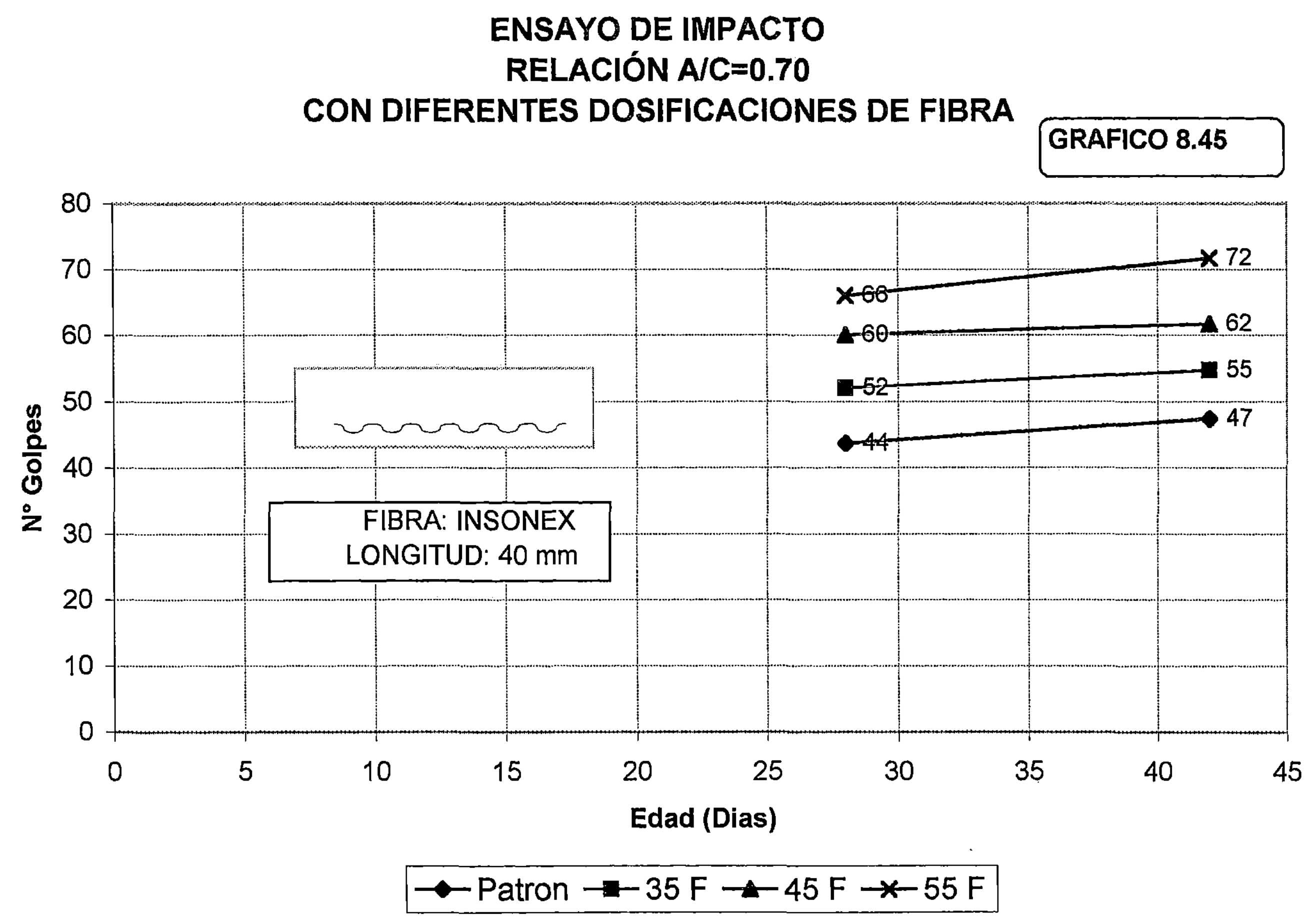
N° Días	Altura		Diámetro		N° Golpes	
	H	Prom	D	Prom	G	Prom
28	6.41	6.50	14.85	14.92	51	52
28	6.65		15.00		49	
28	6.45		14.90		56	
42	6.50	6.53	14.90	14.97	48	55
42	6.45		15.00		60	
42	6.65		15.00		56	

o DOSIFICACIÓN DE LA FIBRA 45 Kg/m^3 DE CONCRETO

N° Días	Altura		Diámetro		N° Golpes	
	H	Prom	D	Prom	G	Prom
28	6.70	6.52	15.00	14.98	61	60
28	6.40		15.05		58	
28	6.45		14.90		61	
42	6.40	6.52	14.80	14.80	58	62
42	6.45		14.80		50	
42	6.70		14.80		77	

o DOSIFICACIÓN DE LA FIBRA 55 Kg/m^3 DE CONCRETO

N° Días	Altura		Diámetro		N° Golpes	
	H	Prom	D	Prom	G	Prom
28	6.60	6.57	15.00	14.93	69	66
28	6.50		15.00		55	
28	6.6		14.80		74	
42	6.60	6.57	14.80	14.83	80	72
42	6.60		14.80		70	
42	6.50		14.90		65	


ENSAYO DE IMPACTO
RELACIÓN A/C=0.70
CON DIFERENTES DOSIFICACIONES DE FIBRA
GRAFICO 8.45
80
70
60
50
40
30
20
10
0
N° Golpes
0
5
10
15
20
25
30
35
40
45
Edad (Dias)
72
66
62
60
55
52
47
44
FIBRA: INSONEX
LONGITUD: 40 mm
Patron
35 F
45 F
55 F

CAPÍTULO 9

ANÁLISIS DE LOS RESULTADOS

9.1 GENERALIDADES

En el presente capitulo se analizara los resultados obtenidos en la presente investigación. Los ensayos realizados en la presente tesis están normados por las Normas Técnicas Peruanas (NTP) antes denominadas ITINTEC.

La presente tesis llamado: "Estudio del comportamiento del concreto de mediana a baja resistencia con la incorporación de Fibras de Acero y cemento Pórtland tipo I Andino" tiene como objetivo realizar la investigación de la variación de las propiedades del concreto para las relaciones agua/cemento de 0.60, 0.65, 0.70; empleando dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto que serán comparados con el concreto patrón. La longitud de las fibras de acero es de 40 mm.

La fibra de acero utilizado para el presente tema de investigación es INSONEX cuya fabricación esta representado por la empresa INSOMIN

Los ensayos realizados en el concreto fueron: En el concreto fresco: Asentamiento, Exudación, Peso Unitario, Tiempo de Fragua, Fluidez, Contenido de Aire; en el concreto endurecido: Resistencia a la Compresión, Tracción por Compresión Diametral, Módulo Elástico Estático, Flexión, Impacto.

El presente capítulo constituye pues la parte más importante de la investigación realizada debido a que del análisis hecho en el mismo, obtendremos las conclusiones y podremos plantear las recomendaciones respectivas.

9.2 ANÁLISIS DE LOS AGREGADOS

La calidad del agregado es de suma importancia, debido a su influencia en el volumen de concreto al ocupar aproximadamente las tres cuartas partes. El agregado no solo puede limitar la resistencia del concreto, sino que sus propiedades pueden afectar enormemente su durabilidad y desempeño. Desde el punto de vista económico, es ventajoso emplear una mezcla con el menor posible de cemento, aunque el costo debe balancearse con las propiedades deseadas del concreto en estado fresco y endurecido.

Un agregado cuyas propiedades resulten satisfactorias hará siempre un buen concreto, pero un agregado de propiedades que se consideran inferiores también podrá lograr la calidad deseada. Por ello es necesario un criterio para el desempeño del concreto, por lo que conviene someterlo a prueba para determinar su valor.

9.2.1 ANÁLISIS DEL AGREGADO FINO

El agregado fino que se utilizó en el presente tema de investigación es proveniente de la cantera "gloria"

Las características físicas del agregado fino con mayor influencia en el desempeño del concreto son : La granulometría, modulo de finura, contenido de humedad, % de absorción y % de finos que pasa la malla N° 200

En el análisis granulométrico realizado al graficar la curva granulométrica podemos observar que esta se encuentra dentro de los limites determinados por la ASTM C-33, esta curva tiene una tendencia hacia una arena gruesa, esto permite que tenga una buena adherencia y trabajabilidad en el concreto.

De hecho no existe una gradación o distribución por tamaños ideal debido a la interacción de los factores que influyen en la manejabilidad; el área de la superficie del agregado, que determina la cantidad de agua necesaria para humedecer todos los sólidos; el volumen relativo ocupado por el agregado; la cantidad de material fino en la mezcla.

El módulo de finura obtenido es 3.01, los valores típicos tienen un rango entre 2.3 y 3.1 donde un valor mas alto indica una gradación mas gruesa. Lo usual es que se calcule el modulo de finura para un agregado fino que para un agregado grueso. La utilidad del modulo de finura radica en la detección de variaciones ligeras en un agregado de la misma fuente, que podrían afectar la manejabilidad del concreto fresco.

El contenido de humedad obtenido es 1.49% y la absorción es 2.96%, que es utilizado en el calculo de las series de cantidades y del requerimiento total del agua de la mezcla, estos valores deben medirse con frecuencia debido a que sufren cambios con el clima, estos ensayos se realizaron en la época de verano.

9.2.2 ANÁLISIS DEL AGREGADO GRUESO

El agregado fino que se utilizó en el presente tema de investigación es proveniente de la cantera "gloria"

En el análisis granulométrico realizado al agregado grueso se puede observar que esta se encuentra dentro de los limites del huso ASTM # 57, debido a que su tamaño nominal varia de 1" a N° 4, el tamaño máximo es de 1" y el tamaño nominal máximo es de 1", esto nos da una idea que un concreto preparado con nuestro agregado grueso puede discurrirse sin dificultad entre los encofrados y los paquetes de varillas colocadas, evitando de esta manera la formación de cavidades llamadas cangrejeras.
El modulo de finura obtenido según ensayo es de 7.13 indicando un valor optimo, que nos permitirá una reducción de agua.

9.2.3 ANÁLISIS DEL AGREGADO GLOBAL

El agregado global es el que se obtiene de la combinación del agregado fino y grueso mediante un porcentaje optimo, obteniendo 51% de agregado fino y 49% de agregado grueso, luego se calcula el modulo de finura utilizando el mismo procedimiento para el agregado fino o grueso, obteniendo un valor de 5.03

La evaluación individual de la granulometría, tanto de la piedra como de la arena no son suficientes, y mas aun cuando se da el caso generalmente de que estos elementos evaluados individualmente, no cumplan con los usos estipulados por la norma ASTM C-33.

Es por ello que logrando una participación porcentual de estos 2 elementos podremos lograr una distribución de partículas. Estas combinaciones se le puede evaluar utilizando curvas teóricas y de husos totales, uno de ellos es el HUSO DIN 1045 el cual se muestra en el cuadro N° 2.12 y la grafica 2.4 , se puede observar en dicha grafica que la curva granulométrica del agregado global tiende a comportarse hacia la parte central de los limites lo cual es aceptable.

9.3 ANÁLISIS COMPARATIVO EN EL CONCRETO FRESCO

9.3.1 ENSAYO DE ASENTAMIENTO (plg)

Dosificación de Fibra Kg/m^3 de Concreto	Asentamiento		
	a/c:0.60	a/c:0.65	a/c:0.70
Patrón (0)	5	5	5
35	3 1/2	3 3/4	4
45	3 1/4	3 1/2	3 3/4
55	3	3 1/4	3 1/2

VARIACIÓN PORCENTUAL CON RESPECTO AL CONCRETO PATRÓN

Dosificación de Fibra Kg/m^3 de Concreto	Porcentaje		
	a/c:0.60	a/c:0.65	a/c.0.70
Patrón (0)	100.00	100.00	100.00
35	70.00	75.00	80.00
45	65.00	70.00	75.00
55	60.00	65.00	70.00

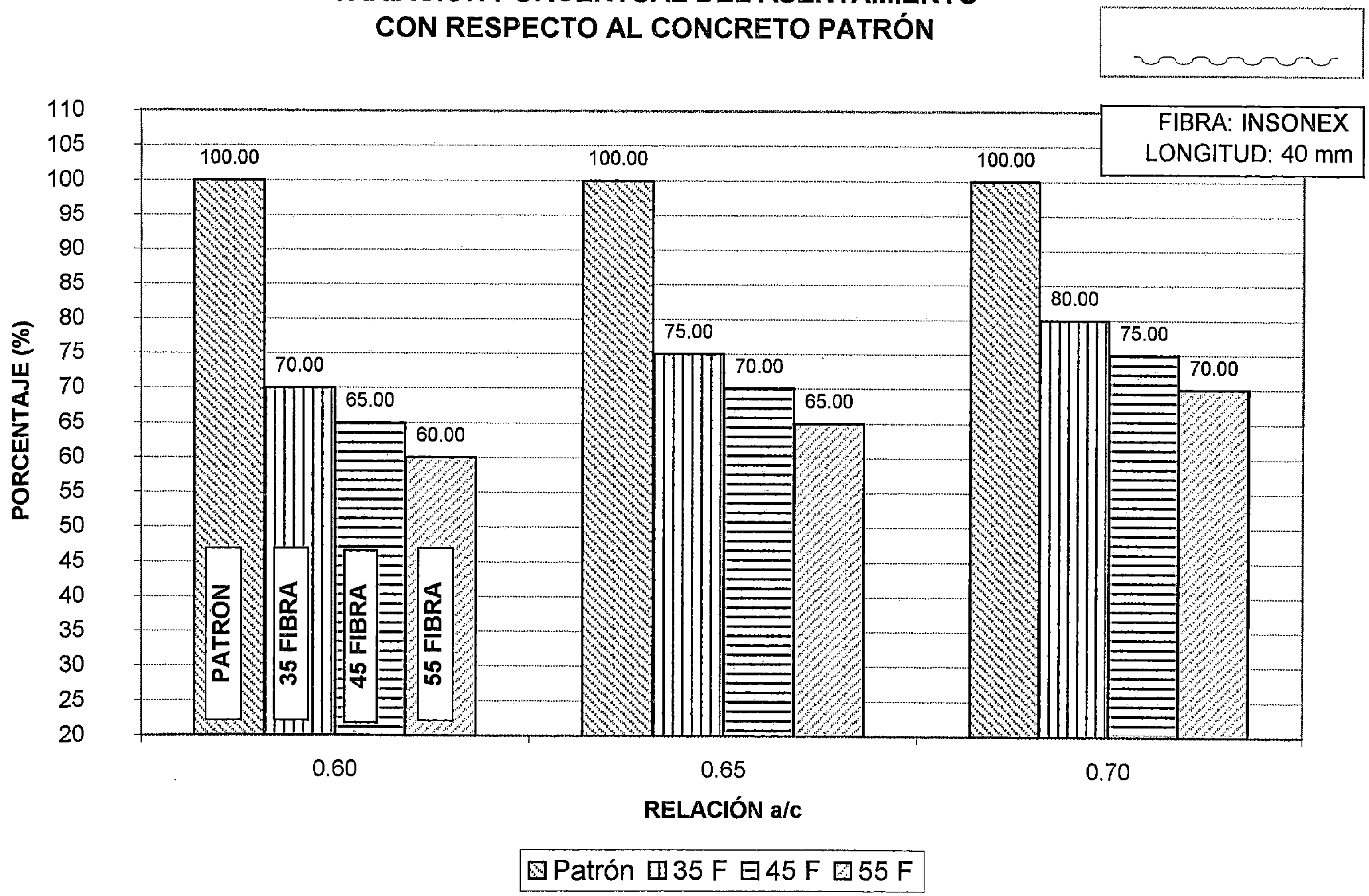
De los cuadros anteriores podemos observar que el Asentamiento en el concreto adicionado con Fibras de Acero disminuye desde un intervalo de 41/2" - 51/2 a un intervalo de 3" – 4" conforme se incrementa la fibra. De esta manera en el cuadro de variación porcentual con respecto al concreto patrón para las relaciones de agua/cemento podemos decir lo siguiente

Relación 0.60 : disminuye en 30%, 35%, 40% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto

Relación 0.65 : disminuye en 25%, 30%, 35% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto

Relación 0.70 : disminuye en 20%, 25%, 30% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto.

VARIACIÓN PORCENTUAL DEL ASENTAMIENTO CON RESPECTO AL CONCRETO PATRÓN



Tesis: "Estudio del comportamiento del concreto de mediana a baja resistencia, con la incorporación de Fibras de Acero y CPT I Andino"

9.3.2 ENSAYO DE EXUDACIÓN (%)

Dosificación de Fibra Kg/m^3 de Concreto	Exudación (%)		
	a/c:0.60	a/c.0.65	a/c:0.70
Patrón (0)	2.58	2.77	2.83
35	2.50	2.70	2.79
45	2.42	2.50	2.53
55	2.30	2.43	2.44

VARIACIÓN PORCENTUAL CON RESPECTO AL CONCRETO PATRÓN

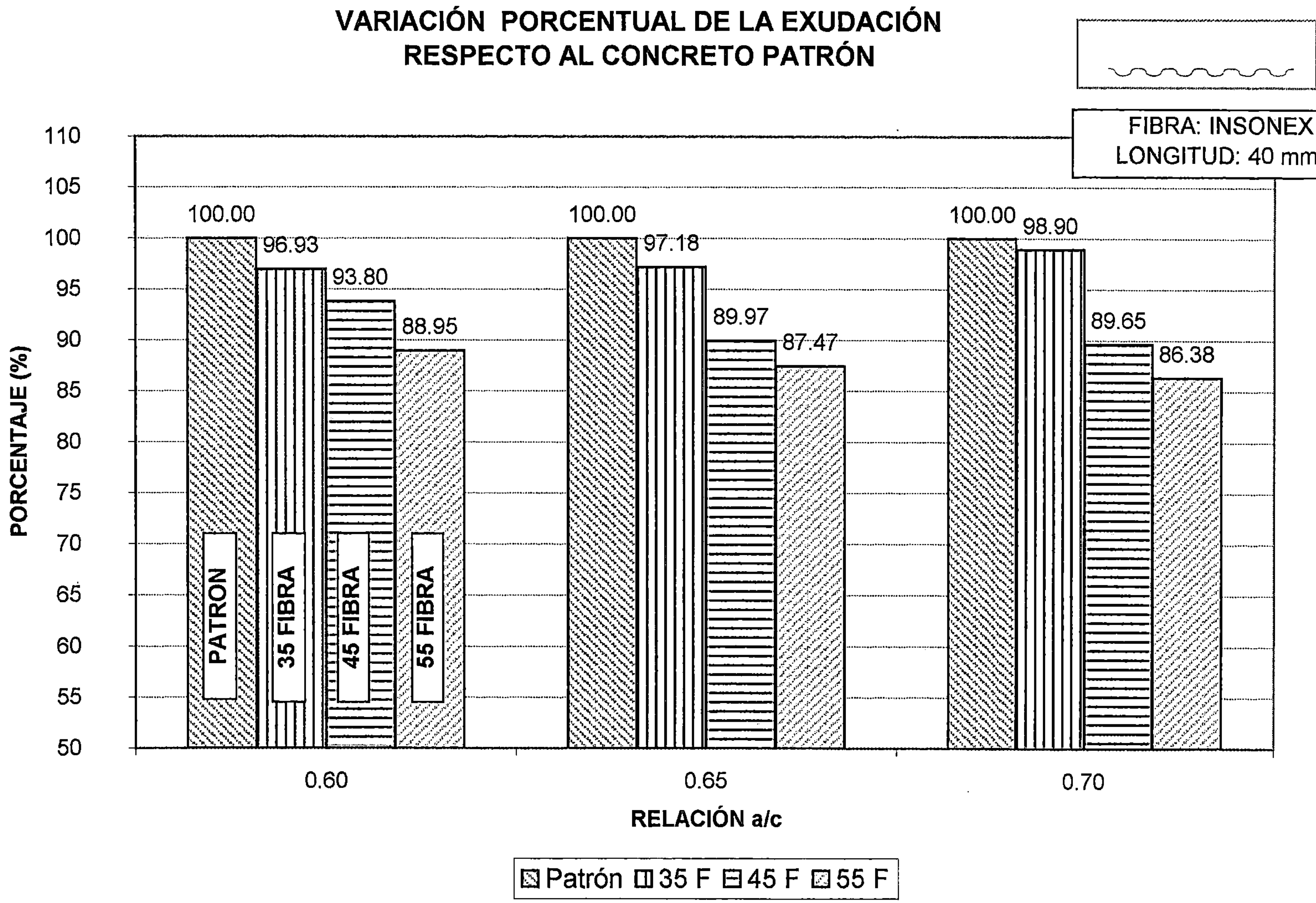
Dosificación de Fibra Kg/m^3 de Concreto	Porcentaje (%)		
	a/c:0.60	a/c:0.65	0.70
Patrón (0)	100.00	100.00	100.00
35	96.93	97.18	98.90
45	93.80	88.97	89.85
55	88.95	87.47	86.38

De los cuadros anteriores podemos observar que la Exudación disminuye conforme se añade fibra de acero al concreto patrón. De esta manera en el cuadro de variación porcentual para las relaciones de agua/cemento decimos:

Relación 0.60 : disminuye en 3.07%, 6.20%, 11.05% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.65 : disminuye en 2.82%, 10.03%, 12.53% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.70 : disminuye en 1.10%, 10.35%, 13.62% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.



Tesis: "Estudio del comportamiento del concreto de mediana a baja resistencia, con la incorporación de Fibras de Acero y CPT I Andino"

9.3.3 ENSAYO DE PESO UNITARIO COMPACTADO (kg/cm^2)

Dosificación de Fibra Kg/m^3 de Concreto	P.U.C. (kg/cm^2)		
	a/c:0.60	a/c.0.65	a/c.0.70
Patrón (0)	2442.86	2450.00	2450.00
35	2446.43	2478.57	2457.14
45	2450.00	2485.71	2492.86
55	2464.29	2492.86	2500.00

VARIACIÓN PORCENTUAL CON RESPECTO AL CONCRETO PATRÓN

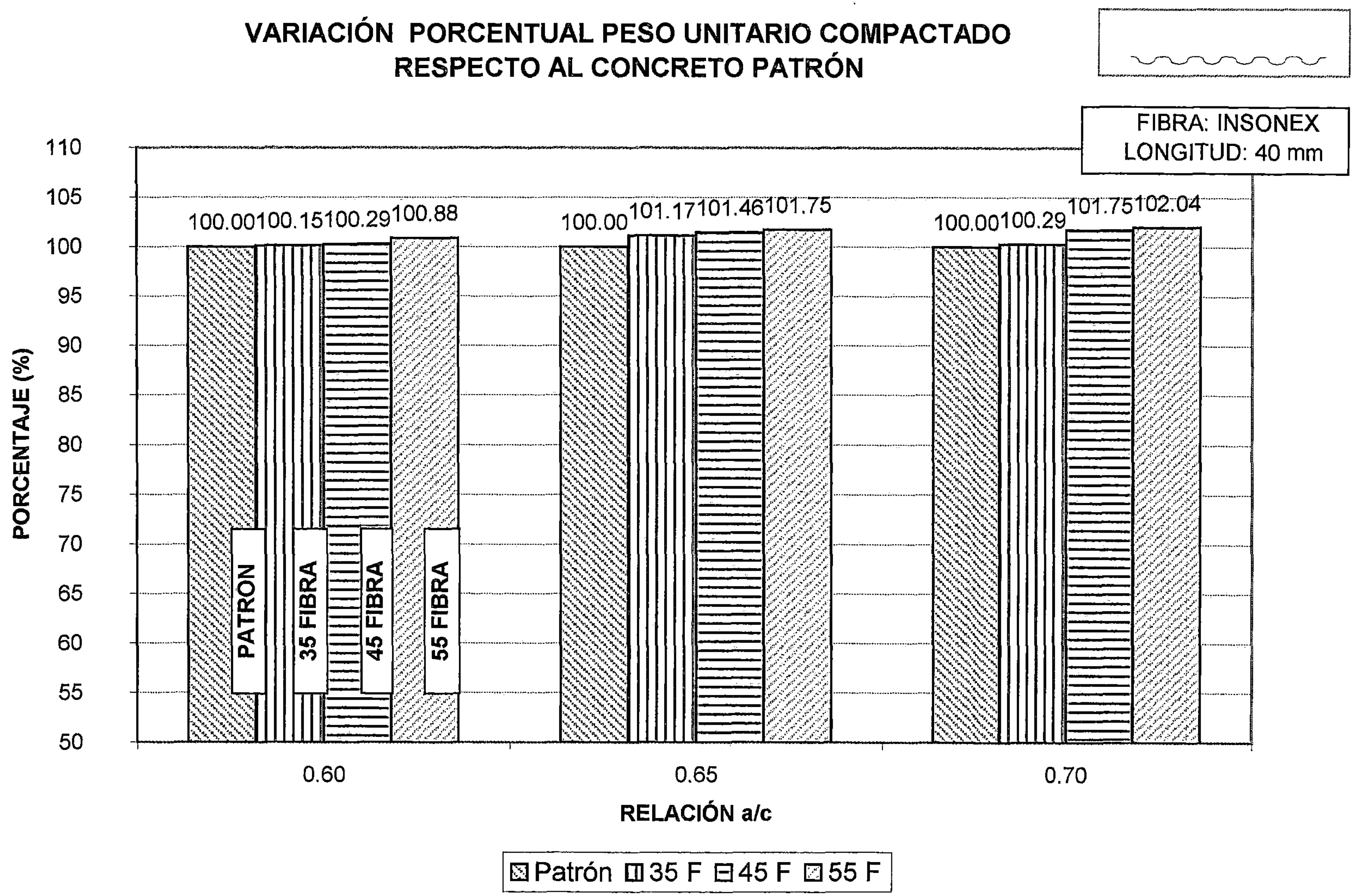
Dosificación de Fibra Kg/m^3 de Concreto	Porcentaje (%)		
	a/c.0.60	a/c.0.65	a/c.0.70
Patrón (0)	100.00	100.00	100.00
35	100.15	101.17	100.29
45	100.29	101.46	101.75
55	100.88	101.75	102.04

De los cuadros anteriores podemos observar que el Peso Unitario Compactado aumenta conforme se añade fibra de acero insonex concreto patrón. De esta manera en el cuadro de variación porcentual para las relaciones de agua/cemento decimos:

Relación 0.60 : aumenta en 0.15%, 0.29%, 0.88% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.65 : aumenta en 1.17%, 1.46%, 1.75% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.70 : aumenta en 0.29%, 1.75%, 2.04% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.



Tesis: "Estudio del comportamiento del concreto de mediana a baja resistencia, con la incorporación de Fibras de Acero y CPT I Andino"

9.3.4 ENSAYO DE TIEMPO DE FRAGUADO (Hr:min)

- **FRAGUADO INICIAL**

Dosificación de Fibra Kg/m^3 de Concreto	Fraguado Inicial (Hr:min)		
	a/c:0.60	a/c:0.65	a/c.0.70
Patrón (0)	04:20	04:30	04:35
35	04:15	04:28	04:30
45	04:11	04:25	04:27
55	04:06	04:20	04:24

VARIACIÓN PORCENTUAL CON RESPECTO AL CONCRETO PATRÓN

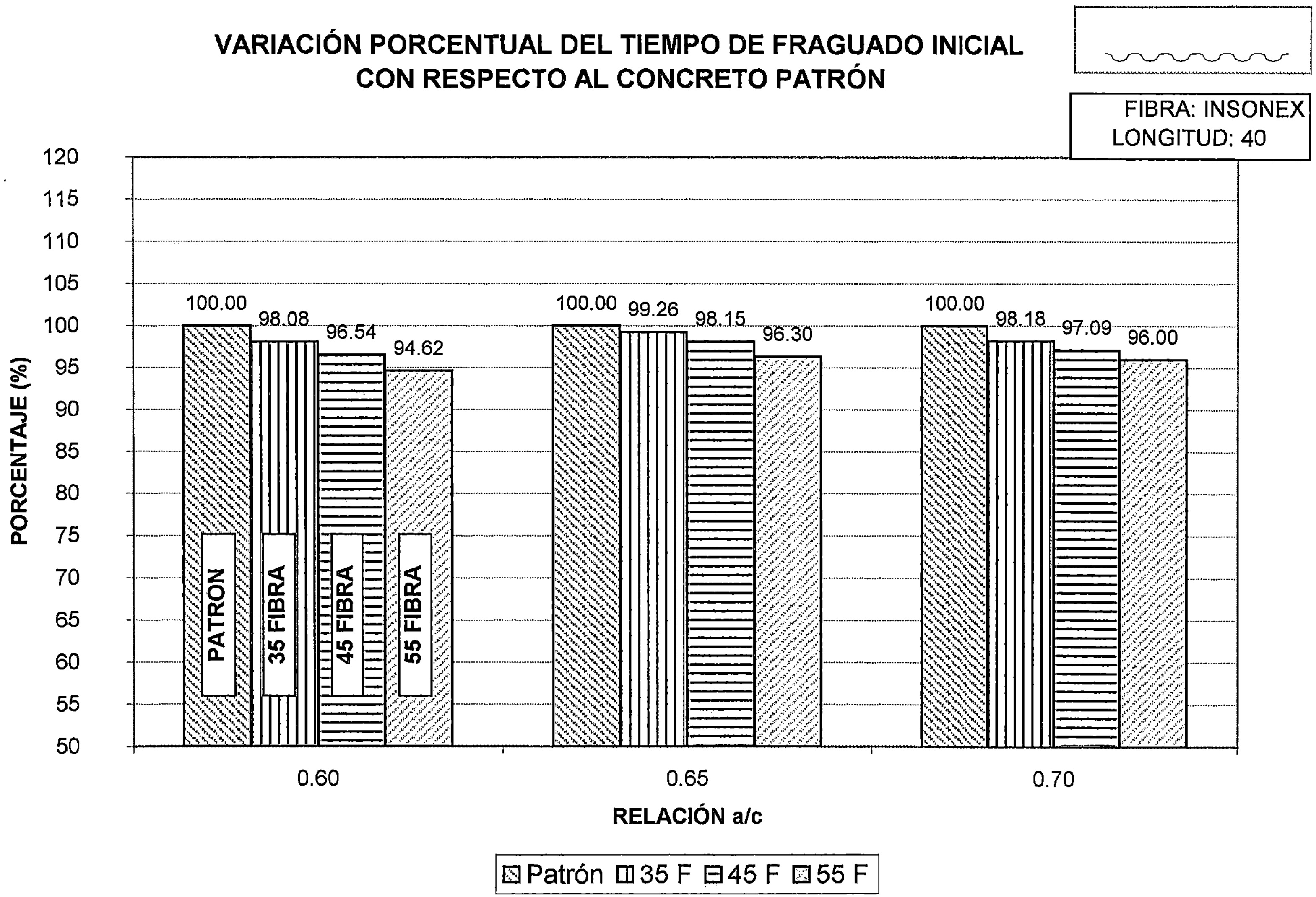
Dosificación de Fibra Kg/m^3 de Concreto	Porcentaje (%)		
	a/c:0.60	a/c:0.65	a/c:0.70
Patrón (0)	100.00	100.00	100.00
35	98.08	99.26	98.18
45	96.54	98.15	97.09
55	94.62	96.30	96.00

De los cuadros anteriores podemos observar que el Tiempo de Fraguado Inicial disminuye conforme se añade fibra de acero insonex al concreto patrón. De esta manera en el cuadro de variación porcentual para las relaciones de agua/cemento decimos:

Relación 0.60 : disminuye en 1.92%, 3.46%, 5.38% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.65 : disminuye en 0.74%, 1.85%, 3.70% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.70 : disminuye en 1.82%, 2.91%, 4.00% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.



VARIACIÓN PORCENTUAL DEL TIEMPO DE FRAGUADO INICIAL
CON RESPECTO AL CONCRETO PATRÓN
FIBRA: INSONEX
LONGITUD: 40
PORCENTAJE (%)
120
115
110
105
100
95
90
85
80
75
70
65
60
55
50
100.00
98.08
96.54
94.62
100.00
99.26
98.15
96.30
100.00
98.18
97.09
96.00
PATRON
35 FIBRA
45 FIBRA
55 FIBRA
0.60
0.65
0.70
RELACIÓN a/c
Patrón
35 F
45 F
55 F

> **FRAGUADO FINAL**

Dosificación de Fibra Kg/m³ de Concreto	Fraguado Final (Hr:min)		
	a/c.0.60	a/c.0.65	a/c.0.70
Patrón (0)	05:25	05:35	05:45
35	05:20	05:32	05:42
45	05:16	05:30	05:36
55	05:10	05:23	05:25

VARIACIÓN PORCENTUAL CON RESPECTO AL CONCRETO PATRÓN

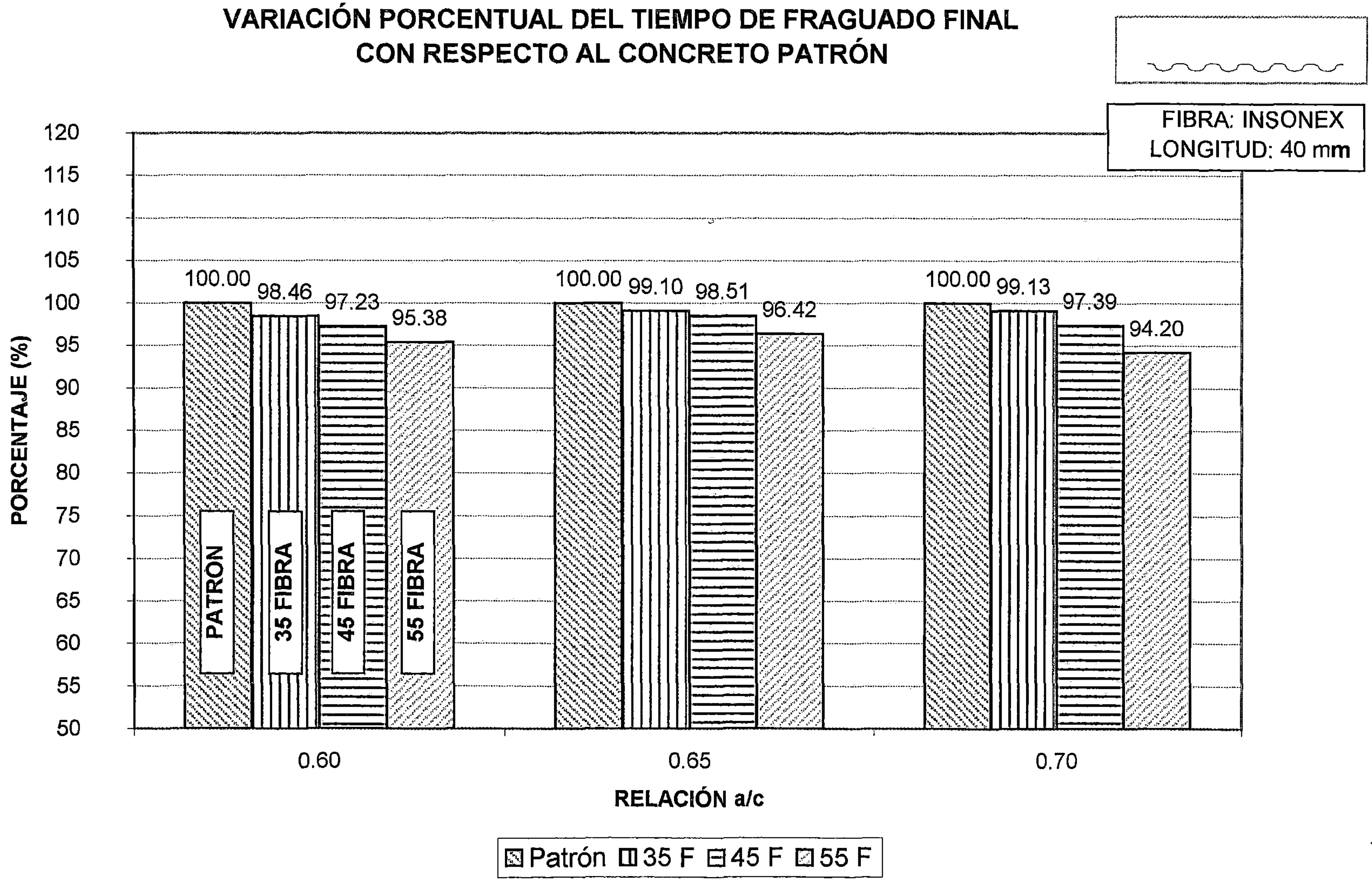
Dosificación de Fibra Kg/m³ de Concreto	Porcentaje (%)		
	a/c:0.60	a/c.0.65	a/c:0.70
Patrón (0)	100.00	100.00	100.00
35	98.46	99.10	99.13
45	97.13	98.51	97.39
55	95.38	96.42	94.20

De los cuadros anteriores podemos observar que el Tiempo de Fraguado Final disminuye conforme se añade fibra de acero insonex al concreto patrón. De esta manera en el cuadro de variación porcentual para las relaciones de agua/cemento decimos:

Relación 0.60 : disminuye en 1.54%, 2.87%, 4.62% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.65 : disminuye en 0.90%, 1.49%, 3.58% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.70 : disminuye en 0.87%, 2.61%, 5.80% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.



Tesis: "Estudio del comportamiento del concreto de mediana a baja resistencia, con la incorporación de Fibras de Acero y CPT I Andino"

9.3.5 ENSAYO DE FLUIDEZ (%)

Dosificación de Fibra Kg/m^3 de Concreto	Fluidez (%)		
	a/c:0.60	a/c:0.65	a/c.0.70
Patrón (0)	105.71	110.83	113.91
35	102.76	106.04	107.35
45	98.82	103.41	102.43
55	93.24	96.19	97.51

VARIACIÓN PORCENTUAL CON RESPECTO AL CONCRETO PATRÓN

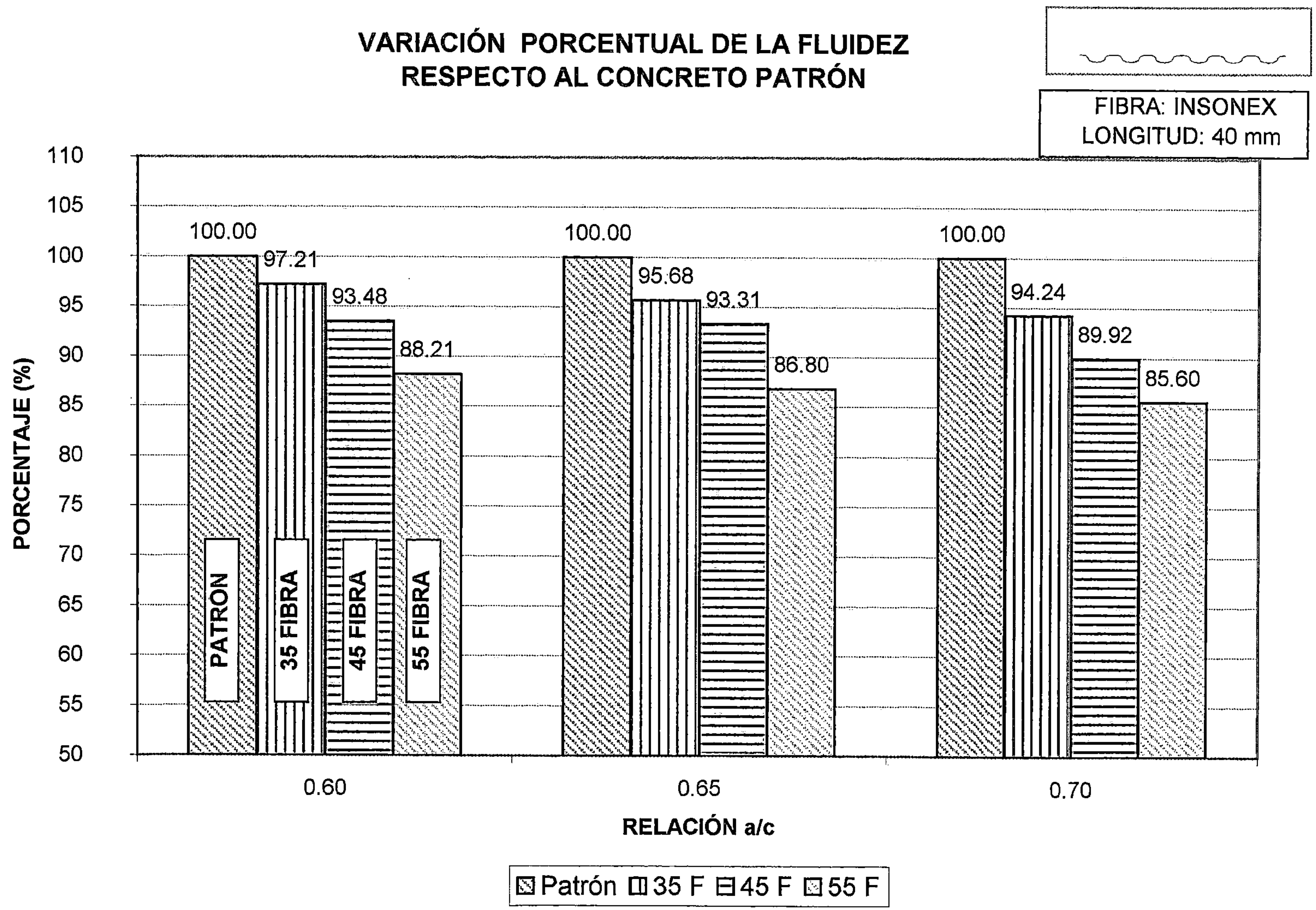
Dosificación de Fibra Kg/m^3 de Concreto	Porcentaje (%)		
	a/c:0.60	a/c:0.65	a/c:0.70
Patrón (0)	100.00	100.00	100.00
35	97.21	95.68	94.24
45	93.48	93.31	89.92
55	88.21	86.80	85.60

De los cuadros anteriores podemos observar que la Fluidez disminuye conforme se añade fibra de acero insonex al concreto patrón. De esta manera en el cuadro de variación porcentual para las relaciones de agua/cemento decimos:

Relación 0.60 : disminuye en 2.79%, 6.52%, 11.79% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.65 : disminuye en 4.32%, 6.69%, 13.20% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.70 : disminuye en 5.76%, 10.08%, 14.40% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.



Tesis: "Estudio del comportamiento del concreto de mediana a baja resistencia, con la incorporación de Fibras de Acero y CPT I Andino"

9.3.6 ENSAYO DE CONTENIDO DE AIRE (%)

Dosificación de Fibra Kg/m³ de Concreto	Contenido de Aire (%)		
	a/c:0.60	a/c:0.65	a/c.0.70
Patrón	1.60	1.50	1.40
35	1.90	1.70	1.50
45	2.10	1.90	1.80
55	2.14	2.05	1.90

VARIACIÓN PORCENTUAL CON RESPECTO AL CONCRETO PATRÓN

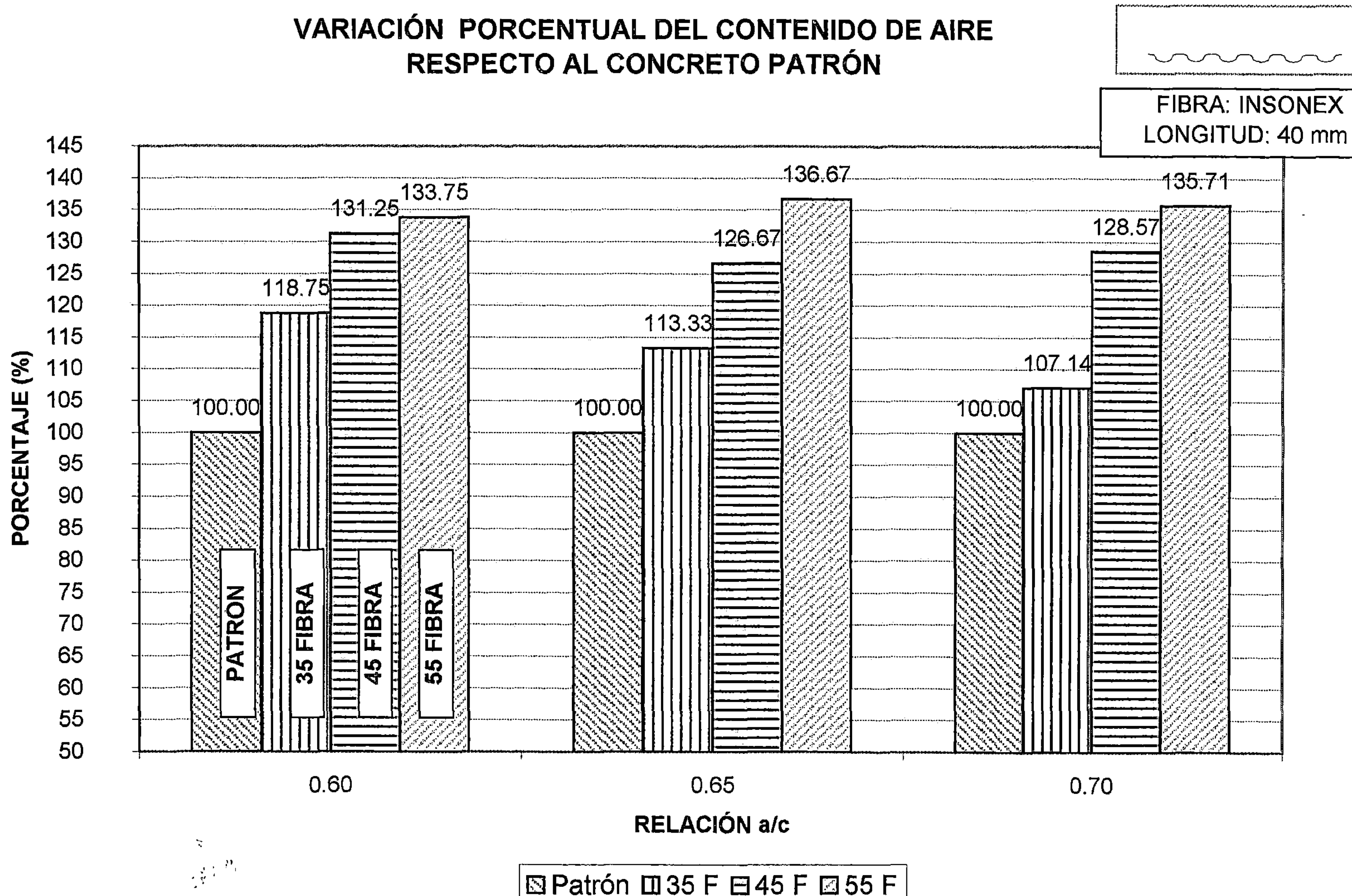
Dosificación de Fibra Kg/m³ de Concreto	Porcentaje (%)		
	a/c.0.60	a/c.0.65	a/c.0.70
Patrón	100.00	100.00	100.00
35	118.75	113.33	107.14
45	131.25	126.67	128.57
55	133.75	136.67	135.71

De los cuadros anteriores podemos observar que el Contenido de Aire aumenta conforme se añade fibra de acero insonex al concreto patrón. De esta manera en el cuadro de variación porcentual para las relaciones de agua/cemento decimos:

Relación 0.60 : aumenta en 18.75%, 31.25%, 33.75% para las dosificaciones de fibra de 35, 45, 55 kg/m³ de concreto respectivamente.

Relación 0.65 : aumenta en 13.33%, 26.67%, 36.67% para las dosificaciones de fibra de 35, 45, 55 kg/m³ de concreto respectivamente.

Relación 0.70 : aumenta en 7.14%, 28.57%, 35.71% para las dosificaciones de fibra de 35, 45, 55 kg/m³ de concreto respectivamente.



VARIACIÓN PORCENTUAL DEL CONTENIDO DE AIRE
RESPECTO AL CONCRETO PATRÓN
FIBRA: INSONEX
LONGITUD: 40 mm
PORCENTAJE (%)
145
140
135
130
125
120
115
110
105
100
95
90
85
80
75
70
65
60
55
50
100.00
118.75
131.25
133.75
100.00
113.33
126.67
136.67
100.00
107.14
128.57
135.71
PATRON
35 FIBRA
45 FIBRA
55 FIBRA
0.60
0.65
0.70
RELACIÓN a/c
Patrón 35 F 45 F 55 F

9.4 ANÁLISIS COMPARATIVO EN EL CONCRETO ENDURECIDO

9.4.1 ENSAYO DE RESISTENCIA A LA COMPRESIÓN (kg/cm²)

- **RELACIÓN AGUA CEMENTO 0.60**

Dosificación de Fibra Kg/m³ de Concreto	Resistencia Compresión (kg/cm²)			
	7 Días	14 Días	28 Días	42 Días
Patrón	183.01	230.82	311.97	333.77
35	207.37	249.12	312.73	334.47
45	212.01	252.78	313.06	335.67
55	224.43	259.28	315.01	336.80

VARIACIÓN PORCENTUAL CON RESPECTO AL CONCRETO PATRÓN

Dosificación de Fibra Kg/m³ de Concreto	Porcentaje (%)			
	7 Días	14 Días	28 Días	42 Días
Patrón (0)	100.00	100.00	100.00	100.00
35	113.31	107.93	100.24	100.21
45	115.84	109.51	100.35	100.57
55	122.63	112.33	100.97	100.91

VARIACIÓN PORCENTUAL CON RESPECTO A LOS 28 DÍAS

Dosificación de Fibra Kg/m³ de Concreto	Porcentaje (%)			
	7 Días	14 Días	28 Días	42 Días
Patrón (0)	58.66	73.99	100.00	106.99
35	66.31	79.66	100.00	106.95
45	67.72	80.75	100.00	107.22
55	71.25	82.31	100.00	106.92

- **RELACIÓN AGUA CEMENTO 0.65**

Dosificación de Fibra Kg/m³ de Concreto	Resistencia Compresión (kg/cm²)			
	7 Días	14 Días	28 Días	42 Días
Patrón (0)	150.62	207.49	258.65	296.00
35	156.88	210.34	259.77	299.46
45	164.73	211.88	264.00	305.40
55	167.19	219.11	269.13	311.25

VARIACIÓN PORCENTUAL CON RESPECTO AL CONCRETO PATRÓN

Dosificación de Fibra Kg/m³ de Concreto	Porcentaje (%)			
	7 Días	14 Días	28 Días	42 Días
Patrón (0)	100.00	100.00	100.00	100.00
35	104.15	101.38	100.43	101.17
45	109.37	102.12	102.07	103.18
55	111.00	105.60	104.05	105.15

VARIACIÓN PORCENTUAL CON RESPECTO A LOS 28 DÍAS

Dosificación de Fibra Kg/m³ de Concreto	Porcentaje (%)			
	7 Días	14 Días	28 Días	42 Días
Patrón (0)	58.23	80.22	100.00	114.44
35	60.39	80.97	100.00	115.28
45	62.40	80.26	100.00	115.68
55	62.12	81.41	100.00	115.65

- **RELACIÓN AGUA CEMENTO 0.70**

Dosificación de Fibra Kg/m³ de Concreto	Resistencia Compresión (kg/cm²)			
	7 Días	14 Días	28 Días	42 Días
Patrón (0)	140.41	198.01	234.90	261.85
35	145.28	203.33	237.73	263.17
45	148.47	207.25	239.90	267.78
55	153.55	213.34	243.70	270.82

VARIACIÓN PORCENTUAL CON RESPECTO AL CONCRETO PATRÓN

Dosificación de Fibra	Porcentaje (%)			
Kg/m^3 de Concreto	7 Días	14 Días	28 Días	42 Días
Patrón (0)	100.00	100.00	100.00	100.00
35	103.47	102.68	101.21	100.51
45	105.75	104.67	102.13	102.27
55	109.36	107.74	103.75	103.43

VARIACIÓN PORCENTUAL CON RESPECTO A LOS 28 DÍAS

Dosificación de Fibra	Porcentaje (%)			
Kg/m^3 de Concreto	7 Días	14 Días	28 Días	42 Días
Patrón (0)	59.77	84.30	100.00	111.47
35	61.11	85.53	100.00	110.70
45	61.89	86.39	100.00	111.62
55	63.01	87.54	100.00	111.13

De los cuadros anteriores podemos observar que la Resistencia a la Compresión aumenta a los 7, 14, 28 y 42 días conforme se añade fibra de acero insonex al concreto patrón.

De esta manera en el primer cuadro de variación porcentual con respecto al concreto patrón para las relaciones de agua/cemento decimos:

Relación 0.60 : aumenta hasta en 22.63% a los 7 días, luego disminuye por encima del concreto patrón alcanzando valores de 12.33% a los 14 días, 0.97% a los 28 días, 0.91% a los 42 días para la dosificación de fibra de 55 kg/m^3 de concreto.

Relación 0.65 : aumenta hasta en 11.00% a los 7 días, luego disminuye por encima del concreto patrón alcanzando valores de 5.60% a los 14 días, 4.05% a los 28 días, 5.15% a los 42 días para la dosificación de fibra de 55 kg/m^3 de concreto

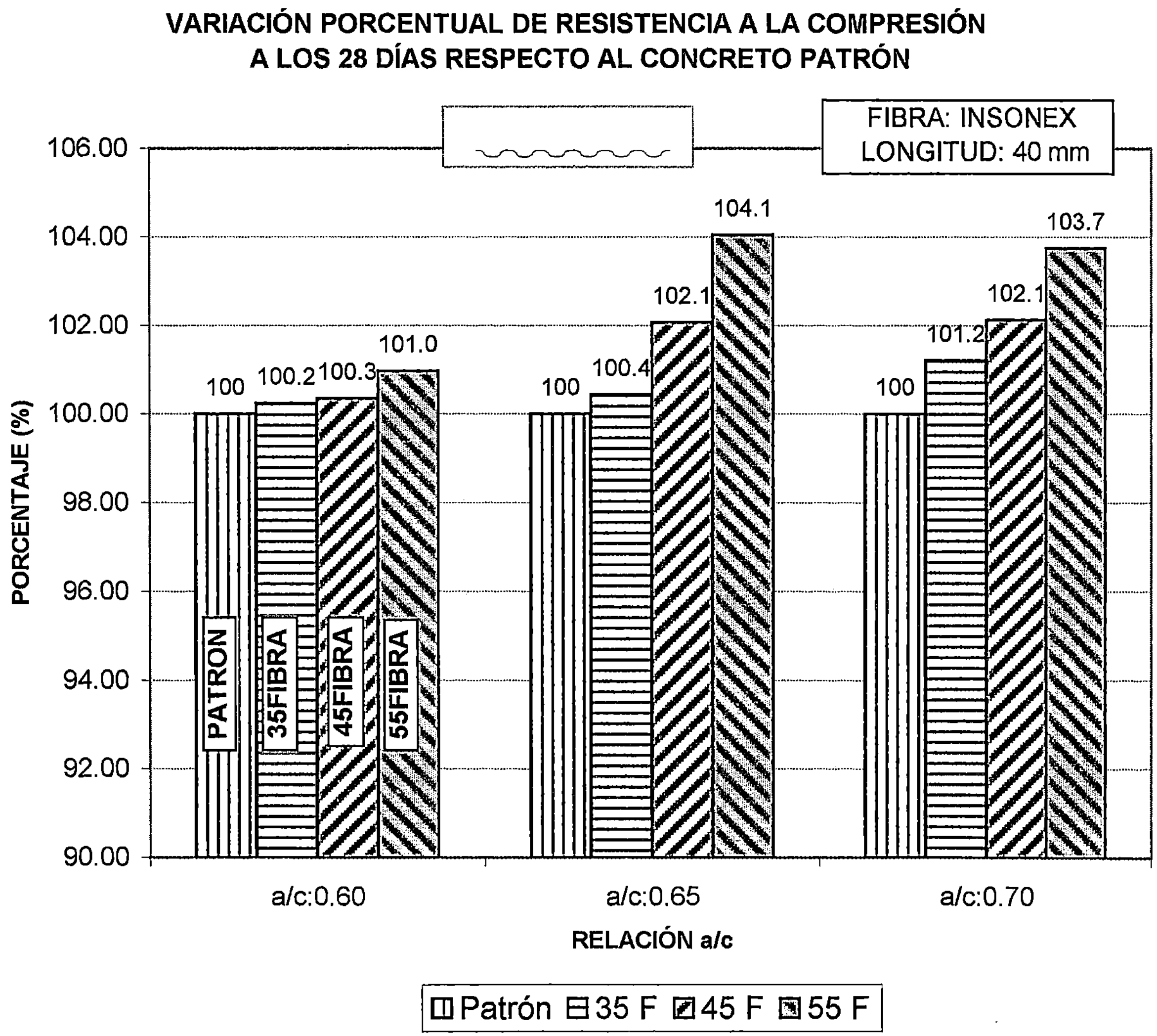
Relación 0.70 : aumenta hasta en 9.36% a los 7 días, luego disminuye por encima del concreto patrón alcanzando valores de 7.74% a los 14 días 3.75% a los 28 días, 3.43% a los 42 días para la dosificación de fibra de 55 kg/m^3 de concreto.

En el segundo cuadro denominado variación porcentual con respecto a los 28 días podemos observar la siguiente tendencia:

Relación 0.60 : El concreto patrón tiene 58.66% a los 7 días, 73.99% a los 14 días y 106.99% a los 42 días; para la dosificación de 35 kg/m^3 de concreto tenemos 66.31% a los 7 días, 79.66% a los 14 días, 106.95% a los 42 días; para la dosificación de 45 kg/m^3 de concreto tenemos 67.72% a los 7 días, 80.75% a los 14 días, 107.22% a los 42 días; para la dosificación de 55 kg/m^3 de concreto tenemos 71.25% a los 7 días, 82.31% a los 14 días, 106.92% a los 42 días.

Relación 0.65 : El concreto patrón tiene 58.23% a los 7 días, 80.22% a los 14 días y 114.44% a los 42 días; para la dosificación de 35 kg/m^3 de concreto tenemos 60.39% a los 7 días, 80.97% a los 14 días, 115.28% a los 42 días; para la dosificación de 45 kg/m^3 de concreto tenemos 62.40% a los 7 días, 80.26% a los 14 días, 115.68% a los 42 días; para la dosificación de 55 kg/m^3 de concreto tenemos 62.12% a los 7 días, 81.41% a los 14 días, 115.65% a los 42 días.

Relación 0.70 : El concreto patrón tiene 59.77% a los 7 días, 84.3% a los 14 días y 111.47% a los 42 días; para la dosificación de 35 kg/m^3 de concreto tenemos 61.11% a los 7 días, 85.53% a los 14 días, 110.70% a los 42 días; para la dosificación de 45 kg/m^3 de concreto tenemos 61.89% a los 7 días, 86.39% a los 14 días, 111.62% a los 42 días; para la dosificación de 55 kg/m^3 de concreto tenemos 63.01% a los 7 días, 87.54% a los 14 días, 111.13% a los 42 días.



Tesis: "Estudio del comportamiento del concreto de mediana a baja resistencia, con la incorporación de Fibras de Acero y CPT I Andino"

9.4.2 ENSAYO DE RESISTENCIA A LA TRACCIÓN POR COMPRESIÓN DIAMETRAL (kg/cm²)

Dosificación de Fibra Kg/m³ de Concreto	Resistencia Tracción C. D. (kg/cm²)		
	a/c:0.60	a/c:0.65	a/c:0.70
Patrón (0)	29.58	25.04	22.73
35	30.36	25.41	24.46
45	32.11	27.51	25.45
55	34.67	29.14	25.70

CUADRO DE VARIACIÓN PORCENTUAL CON RESPECTO AL CONCRETO PATRÓN

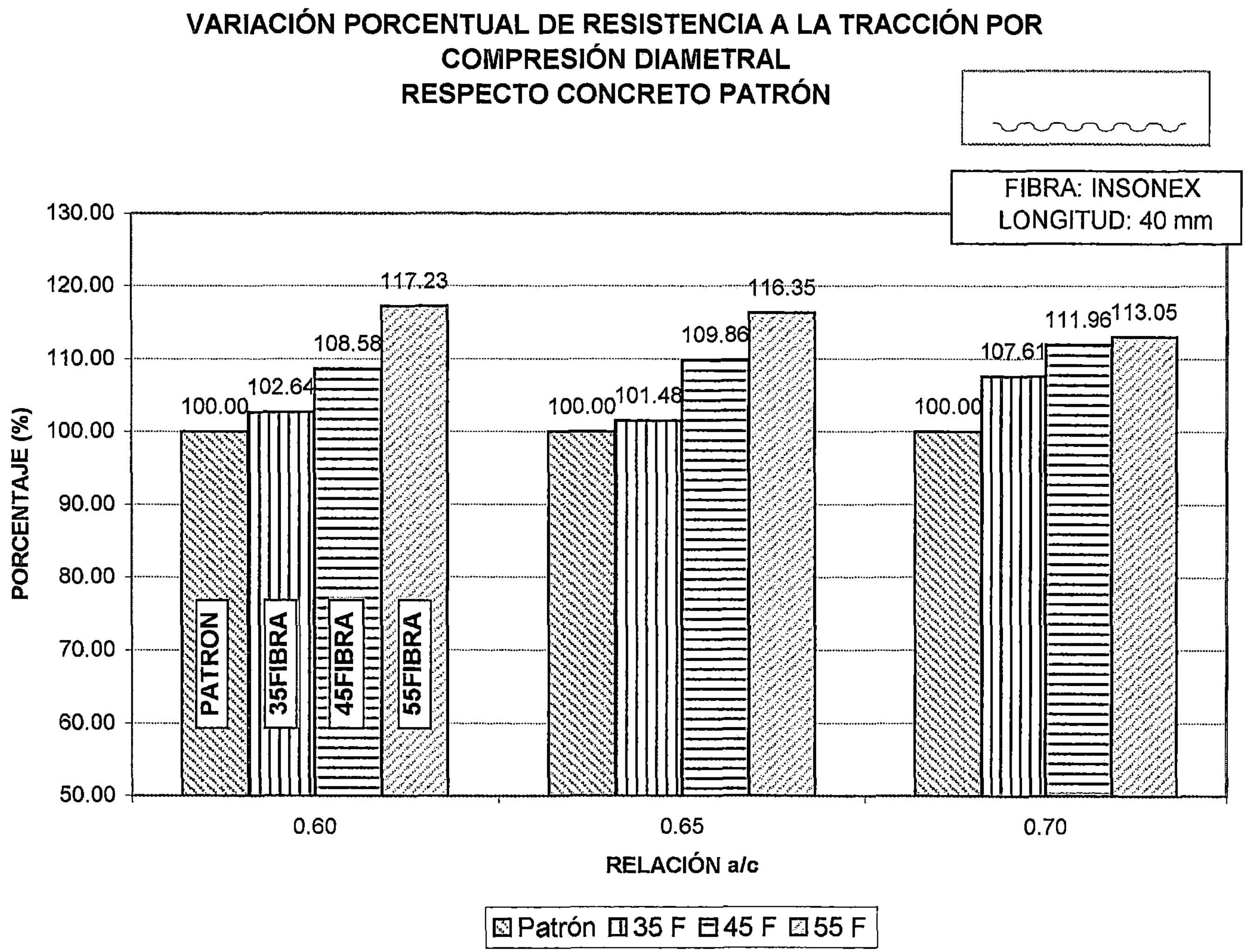
Dosificación de Fibra Kg/m³ de Concreto	Porcentaje (%)		
	a/c.0.60	a/c.0.65	a/c:0.70
Patrón (0)	100.00	100.00	100.00
35	102.64	101.48	107.61
45	108.58	109.86	111.97
55	117.23	116.37	113.07

De los cuadros anteriores podemos observar que la Resistencia a la Tracción por Compresión Diametral aumenta conforme se añade fibra de acero insonex al concreto patrón. De esta manera en el cuadro de variación porcentual para las relaciones de agua/cemento decimos:

Relación 0.60 : aumenta en 2.64%, 8.58%, 17.23% para las dosificaciones de fibra de 35, 45, 55 kg/m³ de concreto respectivamente.

Relación 0.65 : aumenta en 1.48%, 9.86%, 16.35% para las dosificaciones de fibra de 35, 45, 55 kg/m³ de concreto respectivamente.

Relación 0.70 : aumenta en 7.61%, 11.96%, 13.05% para las dosificaciones de fibra de 35, 45, 55 kg/m³ de concreto respectivamente.


VARIACIÓN PORCENTUAL DE RESISTENCIA A LA TRACCIÓN POR COMPRESIÓN DIAMETRAL RESPECTO CONCRETO PATRÓN
FIBRA: INSONEX
LONGITUD: 40 mm
PORCENTAJE (%)
130.00
120.00
110.00
100.00
90.00
80.00
70.00
60.00
50.00
100.00
102.64
108.58
117.23
100.00
101.48
109.86
116.35
100.00
107.61
111.96
113.05
PATRON
35FIBRA
45FIBRA
55FIBRA
0.60
0.65
0.70
RELACIÓN a/c
Patrón
35 F
45 F
55 F

9.4.3 ENSAYO DE MÓDULO DE ELASTICIDAD ESTÁTICO (10^5 kg/cm^2)

Dosificación de Fibra Kg/m^3 de Concreto	Módulo de E. E. (10^5 kg/cm^2)		
	a/c:0.60	a/c.0.65	a/c.0.70
Patrón (0)	2.8525	2.7014	2.5656
35	2.8990	2.7980	2.6040
45	2.9670	2.8340	2.6570
55	3.0900	2.9110	2.7560

CUADRO DE VARIACIÓN PORCENTUAL CON RESPECTO AL CONCRETO PATRÓN

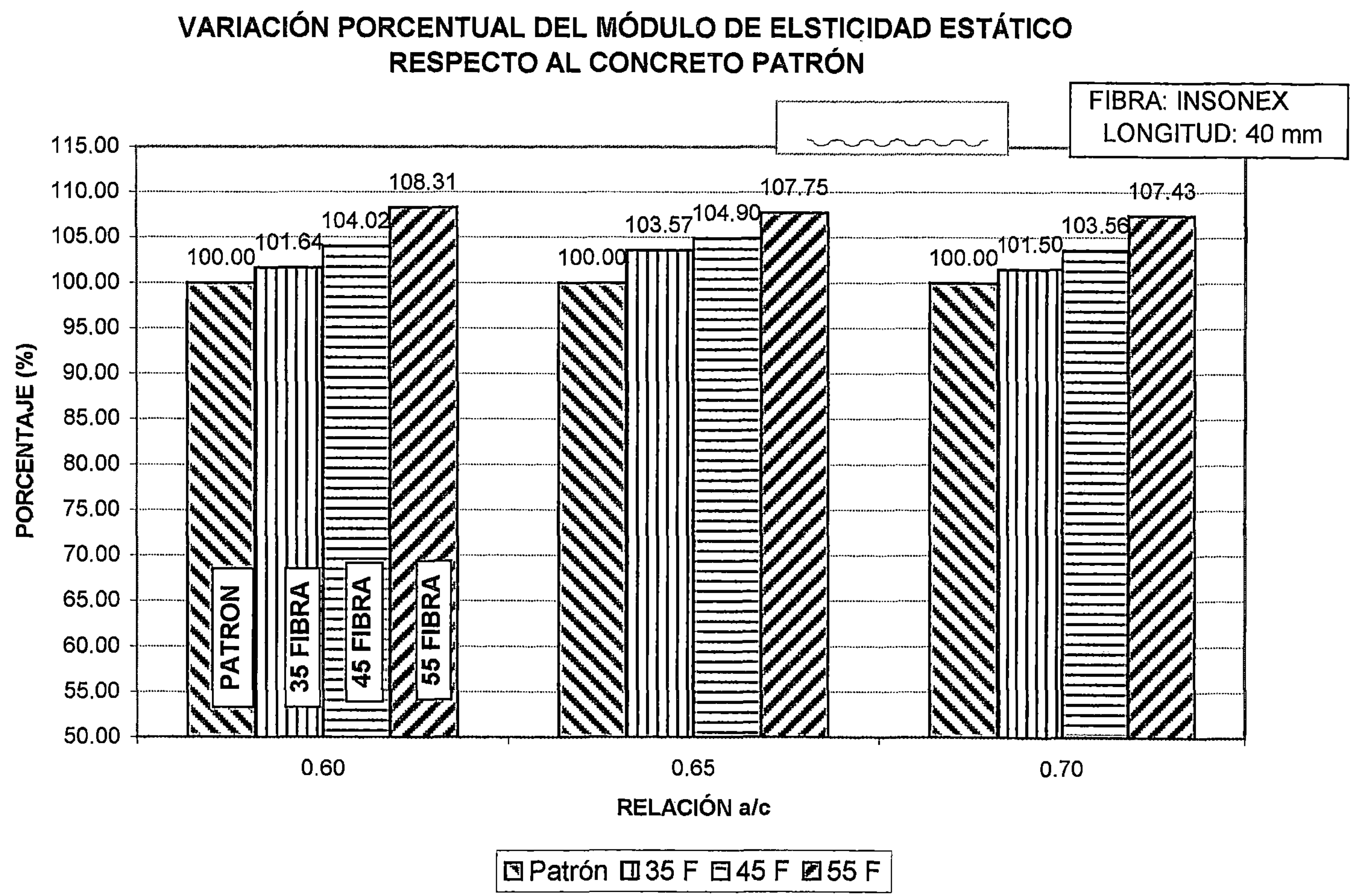
Dosificación de Fibra Kg/m^3 de Concreto	Porcentaje (%)		
	a/c:0.60	a/c.0.65	a/c.0.70
Patrón (0)	100.00	100.00	100.00
35	101.64	103.57	101.50
45	104.62	104.90	103.56
55	108.31	107.75	107.43

De los cuadros anteriores podemos observar que el Módulo de Elasticidad Estático aumenta conforme se añade fibra de acero insonex al concreto patrón. De esta manera en el cuadro de variación porcentual para las relaciones de agua/cemento decimos:

Relación 0.60 : Aumenta en 1.64%, 4.62%, 8.31% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.65 : Aumenta en 3.57%, 4.90%, 7.75% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.70 : Aumenta en 1.50%, 3.56%, 7.43% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.


VARIACIÓN PORCENTUAL DEL MÓDULO DE ELSTICIDAD ESTÁTICO
RESPECTO AL CONCRETO PATRÓN
FIBRA: INSONEX
LONGITUD: 40 mm
PORCENTAJE (%)
115.00
110.00
105.00
100.00
95.00
90.00
85.00
80.00
75.00
70.00
65.00
60.00
55.00
50.00
100.00
101.64
104.02
108.31
100.00
103.57
104.90
107.75
100.00
101.50
103.56
107.43
PATRON
35 FIBRA
45 FIBRA
55 FIBRA
0.60
0.65
0.70
RELACIÓN a/c
Patrón
35 F
45 F
55 F

9.4.4 ENSAYO DE RESISTENCIA A LA FLEXIÓN EN VIGAS (kg/cm^2)

Dosificación de Fibra Kg/m^3 de Concreto	Resistencia a la Flexión (kg/cm^2)		
	a/c:0.60	a/c.0.65	a/c.0.70
Patrón (0)	41.15	38.35	35.54
35	43.89	40.38	36.63
45	45.78	41.11	37.62
55	48.88	43.85	39.86

CUADRO DE VARIACIÓN PÓRCENTUAL CON RESPECTO AL CONCRETO PATRÓN

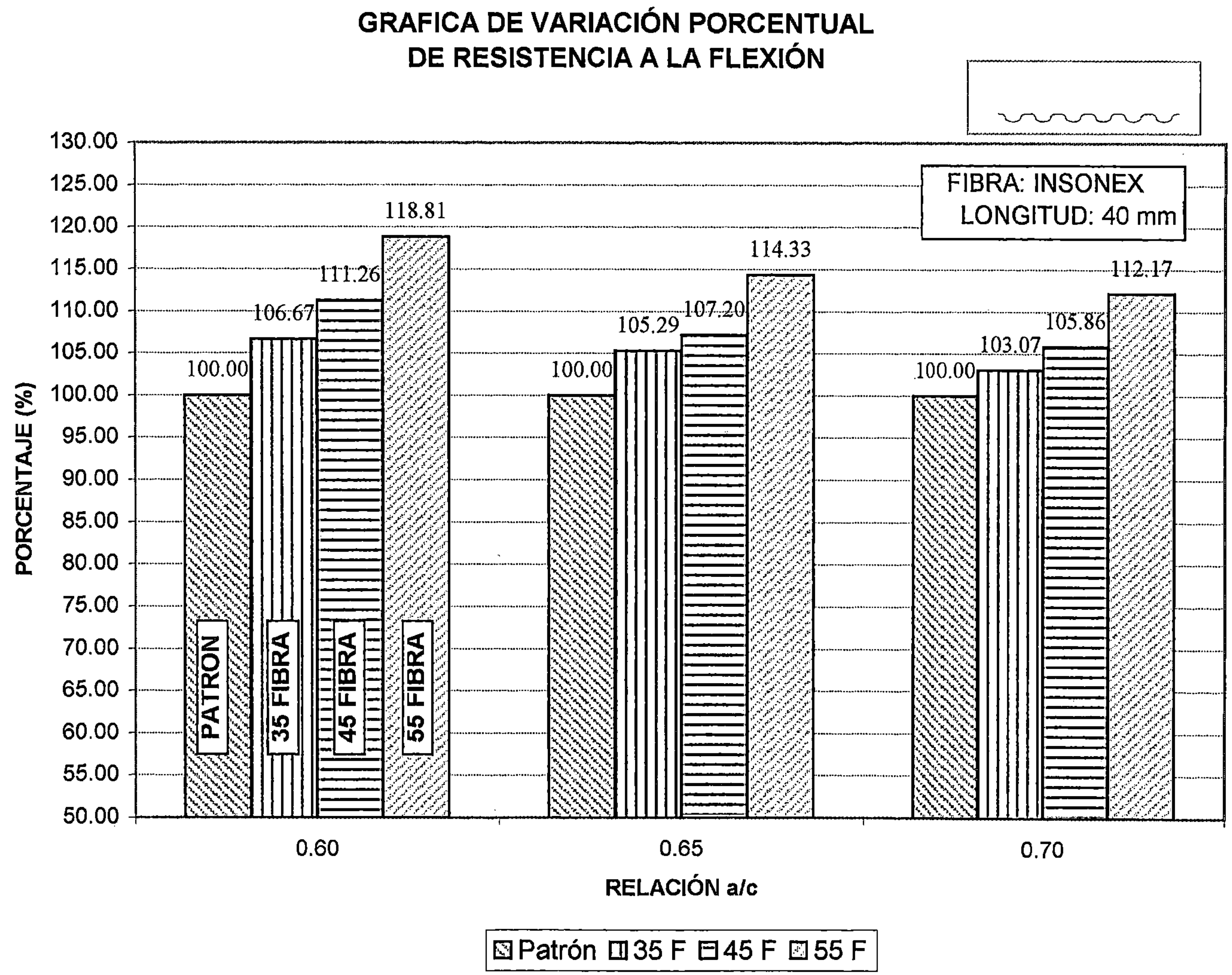
Dosificación de Fibra Kg/m^3 de Concreto	Porcentaje (%)		
	a/c:0.60	a/c.0.65	a/c.0.70
Patrón (0)	100.00	100.00	100.00
35	106.67	105.29	103.07
45	1112.60	107.20	105.86
55	118.81	114.33	112.17

De los cuadros anteriores podemos observar que la Resistencia a la Flexión aumenta conforme se añade fibra de acero insonex al concreto patrón. De esta manera en el cuadro de variación porcentual para las relaciones de agua/cemento decimos:

Relación 0.60 : aumenta en 6.67%, 11.26%, 19.81% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.65 : aumenta en 5.29%, 7.20%, 14.33% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.70 : aumenta en 3.07%, 5.86%, 12.17% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.


GRAFICA DE VARIACIÓN PORCENTUAL
DE RESISTENCIA A LA FLEXIÓN
FIBRA: INSONEX
LONGITUD: 40 mm
PORCENTAJE (%)
130.00
125.00
120.00
115.00
110.00
105.00
100.00
95.00
90.00
85.00
80.00
75.00
70.00
65.00
60.00
55.00
50.00
100.00
106.67
111.26
118.81
100.00
105.29
107.20
114.33
100.00
103.07
105.86
112.17
PATRON
35 FIBRA
45 FIBRA
55 FIBRA
0.60
0.65
0.70
RELACIÓN a/c
Patrón
35 F
45 F
55 F

9.4.4 ENSAYO DE RESISTENCIA AL IMPACTO (N° Golpes)

Relación a/c	Dosificación de Fibra Kg/m^3 de Concreto	R. Impacto (N°Golpes)	
		28 Días	42 Días
0.60	Patrón (0)	63	66
	35	72	99
	45	87	130
	55	107	135
0.65	Patrón (0)	54	58
	35	58	69
	45	61	77
	55	70	79
0.70	Patrón (0)	44	47
	35	52	55
	45	60	62
	55	66	72

CUADRO DE VARIACIÓN PORCENTUAL CON RESPECTO AL CONCRETO PATRÓN

Relación a/c	Dosificación de Fibra Kg/m^3 de Concreto	Porcentaje (%)	
		28 Días	42 Días
0.60	Patrón (0)	100	100
	35	114	150
	45	138	197
	55	170	205
0.65	Patrón (0)	100	100
	35	107	119
	45	113	133
	55	130	136
0.70	Patrón (0)	100	100
	35	118	117
	45	136	132
	55	150	153

De los cuadros anteriores podemos observar que la Resistencia al Impacto aumenta conforme se añade fibra de acero insonex al concreto patrón. De esta manera en el cuadro de variación porcentual para las relaciones de agua/cemento tiene la siguiente tendencia:

A los 28 días:

Relación 0.60 : aumenta en 14%, 38%, 70% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.65 : aumenta en 7%, 13%, 30% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

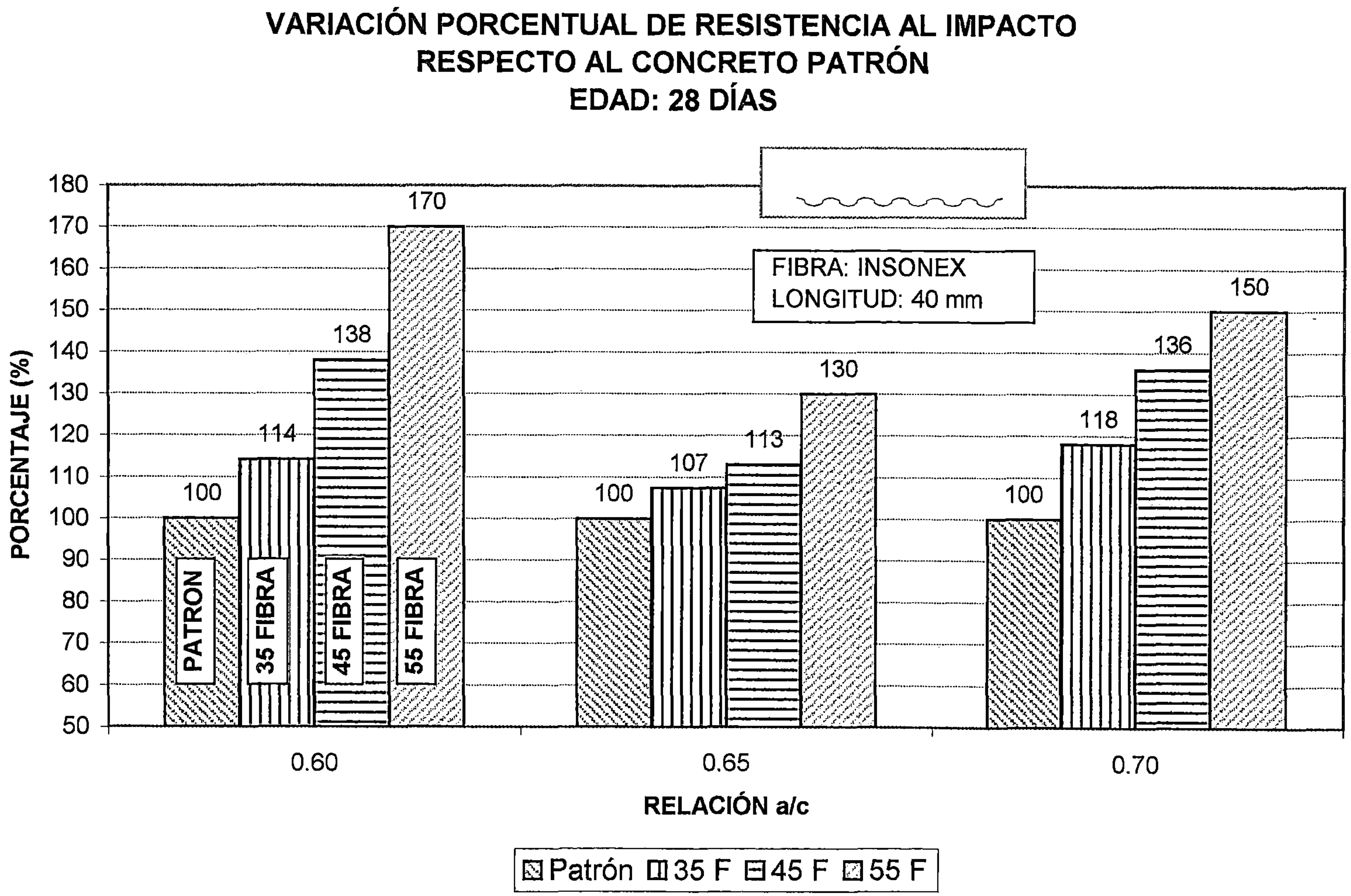
Relación 0.70 : aumenta en 18%, 36%, 50% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

A los 42 días:

Relación 0.60 : aumenta en 50%, 97%, 105% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.65 : aumenta en 19%, 33%, 36% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.

Relación 0.70 : aumenta en 17%, 32%, 53% para las dosificaciones de fibra de 35, 45, 55 kg/m^3 de concreto respectivamente.


VARIACIÓN PORCENTUAL DE RESISTENCIA AL IMPACTO
RESPECTO AL CONCRETO PATRÓN
EDAD: 28 DÍAS
FIBRA: INSONEX
LONGITUD: 40 mm
PORCENTAJE (%)
180
170
160
150
140
130
120
110
100
90
80
70
60
50
100
114
138
170
100
107
113
130
100
118
136
150
PATRON
35 FIBRA
45 FIBRA
55 FIBRA
0.60
0.65
0.70
RELACIÓN a/c
Patrón
35 F
45 F
55 F

CAPÍTULO 10

CONCLUSIONES Y RECOMENDACIONES

10.1 GENERALIDADES

El presente tema de investigación denominado "Estudio del comportamiento del concreto de mediana a baja resistencia, con la incorporación de fibras de acero y Cemento Pórtland Tipo I Andino", estudia al concreto en el estado fresco y endurecido que serán desarrolladas en el concreto patrón (sin fibras) y concreto con fibra de acero con dosificaciones de 35, 45 ,55 kg/m^3 de concreto; para las relaciones de agua/cemento : 0.60, 0.65, 0.70, para luego poder ser comparadas.

Las fibras de acero que se han utilizado para el estudio, es INSONEX, que esta representado por la empresa peruana INSOMIN, esta fibra de acero tiene una geometría ondulada con una longitud de 40 mm, diámetro 0.8 mm, longitud de onda 5mm, altura de la onda 0.65 mm, resistencia mínima a la tracción del alambre 76.5 kg/mm^2, cuya forma de suministro es en cajas de 40 kl.

Las fibras de acero son elementos artificiales que se introducen en la mezcla del concreto como un refuerzo para mejorar algunas de sus propiedades, por le que se realizaron los siguientes ensayos:

EN ESTADO FRESCO:

Ensayo de Asentamiento (plg)
Ensayo de Fluidez (%)
Ensayo de Peso Unitario Compactado (kc/m^3)
Ensayo de Tiempo de Fraguado: Inicial y Final (Hr:min)
Ensayo de Contenido de Aire (%)
Ensayo de Exudación (%)

EN ESTADO ENDURECIDO

Ensayo de Resistencia a la Compresión (kg/cm^2)
Ensayo de Resistencia a la Tracción por Compresión Diametral (kg/cm^2)
Ensayo de Modulo de Elasticidad Estático (10^5 kg/cm^2)
Ensayo de Resistencia a la Flexión (kg/cm^2)
Ensayo de Resistencia al Impacto (N° Golpes)

10.2 CONCLUSIONES

1) El **Asentamiento** del concreto patrón (sin fibras) en el estado fresco **disminuye** al conforme se aumenta la dosificación de fibras de acero Insonex, desde un intervalo de (4 ½" – 5 ½") a un intervalo de (3"– 4"), así tenemos que la variación con respecto al concreto patrón es hasta **40%, 35%, 30%**, para las relaciones de agua/cemento: **0.60, 0.65, 0.70** respectivamente.

2) La **Exudación** del concreto patrón (sin fibras) en el estado fresco **disminuye** conforme se aumenta la dosificación de fibras de acero insonex, así tenemos que la variación con respecto al concreto patrón es hasta **11%, 12%, 13%**, para las relaciones de agua/cemento: **0.60, 0.65, 0.70** respectivamente.

3) El **Peso Unitario Compactado** del concreto patrón (sin fibra) en el estado fresco se **incrementa** conforme se aumenta la dosificación de fibras de acero insonex, así tenemos que la variación con respecto al concreto patrón es hasta **0.8%, 1.7%, 2%**, para las relaciones de agua/cemento: **0.60, 0.65, 0.70** respectivamente.

4) El **Tiempo de Fraguado** del concreto patrón (sin fibras) en el estado fresco, **disminuye** conforme se aumenta la dosificación de fibras de acero insonex, así tenemos que la variación con respecto al concreto patrón del Fraguado Inicial es hasta **5%, 4%, 4%**, y en el Fraguado Final **4%, 4%, 6%** para las relaciones de agua/cemento: **0.60, 0.65, 0.70** respectivamente

5) La **Fluidez** del concreto patrón (sin fibras) en el estado fresco, **disminuye** conforme se aumenta la dosificación de fibras de acero, así tenemos que la variación con respecto al concreto patrón es hasta **11%, 13%, 14%** para las relaciones de agua/cemento: 0.60, 0.65, 0.70 respectivamente.

6) El **Contenido de Aire** del concreto patrón (sin fibras) en el estado fresco, se **incrementa** conforme se aumenta la dosificación de fibras de acero insonex, así tenemos que la variación con respecto al concreto patrón es hasta **33%, 36%, 35%** para las relaciones de agua/cemento: 0.60, 0.65, 0.70 respectivamente.

7) La **Resistencia a la Compresión** del concreto con fibras de acero insonex en el estado endurecido tiene **mayor** resistencia a los 7, 14, 28 y 42 días que el concreto patrón, alcanzando valores de incremento de **23%, 7%, 9%** a la edad de 7 días, para las relaciones de agua/cemento: **0.60, 0.65, 0.70** respectivamente.

8) La **Resistencia a la Tracción por compresión diametral** del concreto patrón (sin fibras) en el estado endurecido, se **incrementa** conforme se aumenta la dosificación de fibras de acero insonex, así tenemos que la variación con respecto al concreto patrón alcanza valores de **17%, 16%, 13%** para las relaciones de agua/cemento: **0.60, 0.65, 0.70** respectivamente.

9) El **Módulo de Elasticidad Estático** del concreto patrón (sin fibra) en el estado endurecido **se incrementa** conforme se aumenta la dosificación de fibra de acero insonex, así tenemos que la variación con respecto al concreto patrón alcanza valores de **8%, 8%, 7%** para las relaciones de agua/cemento: **0.60, 0.65, 0.70** respectivamente.

10) La **Resistencia a la Flexión en vigas** del concreto patrón (sin fibras) en el estado endurecido se **incrementa** conforme se aumenta la dosificación de fibras de acero insonex, así tenemos que la variación con respecto al concreto patrón alcanza valores de **20%, 14%, 12%** para las relaciones agua/cemento: **0.60, 0.65, 0.70** respectivamente,Así mismo se pudo observar que la falla no es brusca en vigas con fibras de acero debido a la presencia de estas en el plano de falla, permitiendo retardar el crecimiento de grietas y aumentar la ductilidad. al transmitir esfuerzo a través de la sección agrietada por lo que es posible un deformación mucho mayor.

11) Al realizar el corte a la viga, la **longitud de la fibra** de acero tuvo un **incremento** de **2.5 mm** (40 mm a 42.5 mm).

12) La **Resistencia al Impacto** en discos del concreto patrón (sin fibras) en el estado endurecido se **incrementa** significativamente conforme se aumenta la dosificación de fibra de acero insonex, así tenemos que a la edad de **28 días** la variación alcanza valores de **70%, 30%, 50%** y a los 42 días **105%, 36, 53%** para las relaciones de agua/cemento: **0.60, 0.65, 0.70** respectivamente.

13) Como conclusión final se puede decir que de acuerdo a los resultados obtenidos en el presente tema de investigación, la relación de agua/cemento: 0.60 con dosificación de fibra de acero de 45 kg/m^3 de concreto se obtienen los principales efectos en el concreto, el cual podemos resumirlos en lo siguiente:.

- Disminuye el Asentamiento (40%).
- Disminuye la Exudación (13).
- Incremento del Peso Unitario Compactado (2%).
- Disminuye el Tiempo de Fraguado Inicial y Final (5%, 6%).
- Disminuye la Fluidez (14).
- Increment0 del contenido de aire (33%).
- Incremento de la resistencia inicial de la compresión (23%).
- Incremento de la resistencia a la Tracción (17%).
- Incremento de la resistencia a la Flexión en vigas (20%).
- Incremento de la resistencia al Impacto (70%).
- Incremento del Módulo de Elasticidad Estático (8%).

14) A continuación se presenta la variación porcentual con respecto al concreto patrón de los ensayos en estado endurecido de la fibra de acero Insonex y Fiberstrand 300 realizado en el LEM de la Universidad Nacional de Ingeniería por el Bachiller Julio Cuaresma Carbajal, para la relación agua/cemento: 0.60.

VARIACIÓN PORCENTUAL DE ENSAYOS CON DIFERENTES FIBRAS						
Relación agua/cemento: 0.60	Fibra de Acero: INSONEX Longitud: 40mm Geometría: Ondulada Dosificación: 45 kg/m^3 concreto			Fibra de Acero:FIBERSTRAN 300 Longitud: 30mm Geometría: Ondulada Dosificación: 40 kg/m^3 concreto		
ENSAYO	Patrón. (sin fibras)	Con Fibra	Variación (Porcentaje)	Patrón. (sin fibras)	Con Fibra	Variación (Porcentaje)
Compresión 28 Días (kg/cm^2)	311.97	304.72	97.68	306.50	293.90	95.89
Flexión en Vigas (kg/cm^2)	41.15	45.78	111.25	37.00	43.81	118.41
Tracción Comp. Diam. (kg/cm^2)	29.58	32.11	108.55	28.60	30.86	107.90
Módulo de E. E. (10^5 kg/cm^2)	2.85	2.50	87.72	2.28	2.19	96.06
Impacto (N° Golpes)	63.00	87.00	138.10	-	-	-

10.3 RECOMENDACIONES

1) Como uno de los problemas fue el proceso de mezclado de las fibras de acero con los componentes del concreto es recomendable adicionarlo en el siguiente orden: agregado grueso, Fibra de Acero, agregado fino, parte de la dosificación del agua, cemento y el restante del agua, para poder lograr una mejor distribución en el concreto evitando de esta manera la formación de grumos. De acuerdo a la preparación del concreto con Fibras realizadas en el presente tema de investigación se recomienda mezclar durante 4 a 5 minutos.

2) El control de la seguridad es un factor importante en el proceso de mezclado por lo que se recomienda el uso de guantes y protección de los ojos, cuando se manejen o añadan las fibras de acero al concreto.

3) Los Agregados cumplen un papel importante debido a su influencia en el volumen del concreto por lo que el estudio de sus características físicas deben realizarse con detenimiento, así mismo se recomienda utilizar un agregado grueso con TM hasta una 1" para que las fibras puedan acomodarse en el concreto, con respecto al agregado fino no es conveniente utilizar arenas finas mayores a las establecidas en las especificaciones por su adherencia en el concreto.

4) De las relaciones de agua/cemento estudiadas se recomienda el uso de fibras de acero hasta una relación de agua/cemento 0.60 es decir concretos de mediana a alta resistencia, con dosificación de fibras de acero de 45 kg/m^3 de concreto, el cual desempeña un mejor comportamiento en los ensayos en el concreto tanto en estado fresco como en estado endurecido, debido a que valores mayores a este 0.65, 0.70 generan en el concreto endurecido disminución en sus propiedades.

5) La geometría de la fibra de acero es un indicativo importante por lo que se recomienda fibras de acero con geometría no muy pronunciada para evitar que estas se concentren en un solo lugar teniendo en cuenta la relación de forma de la fibra de acero es decir, relación de la longitud al diámetro medio.

6) La fibra de acero debe protegerse de agentes externos como lluvia, humedad, etc manteniéndolo almacenado en su caja.

7) Se recomienda el estudio del comportamiento del mortero con fibras de acero

8) En este estudio no se ha considerado la influencia que podría tener la longitud de la fibra de acero, parámetro que sería conveniente tenerla en consideración para futuros estudios.

9) Se recomienda el ensayo de flexión compuesta (en apoyo continuo perimetral), cuya probeta sería una losa plana con dimensiones de sección libre de 50x50 cms con espesor de 10 cm., con carga aplicada en el centro de la sección repartida en dado de 10 cms

10) El uso de fibras de acero incrementa esencialmente las propiedades del concreto en estado endurecido, la manera más fácil y económica de mejorarla es optimizando el contenido de cemento, la relación agua/cemento, la naturaleza de los agregados y la forma de compactar y curar el concreto.

CAPÍTULO 11

ANEXOS

ANEXO A: TABLAS GRANULOMETRICAS DE LOS AGREGADOS

TABLA N° 1

GRANULOMETRIA DEL AGREGADO FINO

Porcentaje de peso (masa) que pasa

TAMIZ		Limites Totales:	C	M	F
9.5	mm (3/8)	100	100	100	100
4.75	mm (No 4)	89 - 100	95 - 100	89 - 100	89 - 100
2.36	mm (No 8)	65 - 100	80 - 100	65 - 100	65 - 100
1.18	mm (No 16)	45 - 100	50 - 85	45 - 100	45 - 100
600	u (No 30)	25 - 100	25 - 60	25 - 100	25 - 100
300	u (No 50)	5 - 70	10 - 30	5 - 70	5 - 70
150	u (No 100)	0 - 12	2 - 10	0 - 12*	0 - 12

* Incrementar a 15% para agregado fino triturado, exepto cuando se use para pavimentos

TABLA N° 2

LIMITES DE SUSTANCIAS DAÑINAS

	Agregado Fino	Agregado Grueso
Particulas delznables, maximo	3%	5%
Material más fino que la Malla ITINTEC 75 m (N° 200), máx.	5%	1%
Carbón y lignito, máximo	0.5%	0.5%
Materia Orgánica	El agregado fino que no demuestre presencia nociva de material orgánica, cuande se determine conforme ITINTEC 400.013, se deberá considerar satisfactorio. El agregado fino que no cumple con el ensayo anterior, podrá ser usado si al determinarse el efecto de las impurezas orgánicas sobre la resistencia de morteros (ITINTEC 400.024) la resistencia relativa a los 7 días no es menor de 95%	

TABLA N° 3

REQUERIMIENTOS DE GRANULOMETRÍA DE LOS AGREGADOS GRUESOS

No ASTM	Tamaño Nominal	% QUE PASA POR LOS TAMICES NORMALIZADOS												
		100 mm (4")	90 mm (3 1/2")	75 mm (3")	63 mm (2 1/2")	50 mm (2")	37.5 mm (1 1/2")	25.0 mm (1")	19.0 mm (3/4")	12.5 mm (1/2")	9.5 mm (3/8")	4.75 mm (No 4)	2.36 mm (No 8)	1.18 mm (No 16)
1	90 a 37.5mm (3 1/2" a 1 1/2")	100	90 a 100		25 a 60		0 a 15		0 a 5					
2	63 a 37.5 mm (2 1/2" a 1 1/2")			100	90 a 100	35 a 70	0 a 15		0 a 5					
3	50 a 25.0 mm (2" a 1")				100	90 a 100	35 a 70	0 a 15		0 a 5				
357	50 a 4.75 mm (2" a No 4)				100	95 a 100		35 a 70		10 a 30		0 a 5		
4	37.5 a 19.0 mm (1 1/2" a 3/4")					100	90 a 100	20 a 55	0 a 15		0 a 5			
467	37.5 a 4.75 mm (1 1/2" a No 4)					100	95 a 100		35 a 70		10 a 30	0 a 5		
5	25 a 12.5 mm 1" a 1/2"						100	90 a 100	20 a 55	0 a 10	0 a 5			
56	25 a 9.5 mm 1" a 3/8"						100	90 a 100	40 a 85	10 a 40	0 a 15	0 a 5		
57	25 a 4.75 mm (1" a No 4)						100	95 a 100		25 a 60		0 a 10	0 a 5	
6	19.0 a 9.5 mm (3/4 " a 3/8")							100	90 a 100	20 a 55	0 a 15	0 a 5		
67	19.0 a 4.75 mm (3/4 " a No 4)							100	90 a 100		20 a 55	0 a 10	0 a 5	
7	12.5 mm a 4.75 mm (1/2" a No 4)								100	90 a 100	40 a 70	0 a 15	0 a 5	
8	9.5 a 2.36 mm (3/8" a No 8)									100	85 a 100	10 a 30	0 a 10	0 a 5

TABLA N° 4

GRANULOMETRIA DEL AGREGADO GLOBAL

TAMIZ		Tamaño Nominal 37.5 mm (1/2")	Tamaño Nominal 19.0 mm (3/4")	Tamaño Nominal 9.5 mm (3/8")
50,0 mm	2"	100		
37.5 mm	(1 1/2")	95 a 100	100	
19.0 mm	(3/4")	45 a 80	95 a 100	
12.5 mm	(1/2")			100
9.5 mm	(3/8")			95 a 100
4.75 mm	(N° 4)	25 a 50	35 a 55	30 a 65
2.36 mm	(N° 8)			20 a 50
1.18 mm	(N° 16)			15 a 40
600 mm	(N° 30)	8 a 30	10 a 35	10 a 30
300 mm	(N° 50)			5 a 15
150 mm	(N° 100)	0 a 8*	0 a 8*	0 a 8*

* Incrementar a 10% para finos de roca triturada

TABLA N° 5

LINEAS GRANULOMETRICAS CONTINUAS

TAMAÑO MAXIMO = 8mm			
MALLA	FRACCION QUE PASA		
(mm)	A	B	C
8.00	100	100	100
4.00	61	74	85
2.00	36	57	71
1.00	21	42	57
0.50			
0.25	5	11	21

TAMAÑO MAXIMO = 16mm			
MALLA	FRACCION QUE PASA		
(mm)	A	B	C
16.00	100	100	100
8.00	60	76	88
4.00	36	56	74
2.00	21	42	62
1.00	12	32	49
0.50			
0.25	3	8	18

TAMAÑO MAXIMO = 32mm			
HUSO DIN (1045)			
MALLA	FRACCION QUE PASA		
(mm)	A	B	C
31.50	100	100	100
16.00	62	80	89
8.00	38	62	77
4.00	23	47	65
2.00	14	37	53
1.00	8	28	42
0.50			
0.25	2	8	15

TAMAÑO MAXIMO = 63mm			
MALLA	FRACCION QUE PASA		
(mm)	A	B	C
63.00	100	100	100
31.50	67	80	90
16.00	46	64	80
8.00	30	50	70
4.00	19	38	59
2.00	11	30	49
1.00	6	24	39
0.50			
0.25	2	7	14

ANEXO B: SECUENCIA DE DISEÑO DE MEZCLA

GENERALIDADES

En el presente anexo se presenta el procedimiento para obtener las dosificaciones requeridas en el diseño de mezcla tanto para el concreto patrón como para el concreto con fibras de acero insonex.

1. Se realiza un diseño de mezcla preliminar para encontrar el porcentaje de arena optimo de la siguiente manera:
   - Se encuentra la cantidad de agua que de un asentamiento de 5" para un porcentaje de arena = 0.47 obtenido mediante el ensayo de peso unitario compactado del agregado global, asumiendo una relación de agua/cemento= 0.65.
   - Con la cantidad de agua encontrada se realiza diseños de mezcla para encontrar la máxima resistencia a la compresión para porcentajes de arena = 0.47, 0.50, 0.53.
   - Se realiza una curva de resistencia a la compresión vs. porcentaje de arena, donde se encuentra que para la máxima resistencia a la compresión le corresponde un %Ar = 0.51.

2. Una vez encontrado el porcentaje de arena optimo se procede a encontrar el agua optima para las relaciones de agua/cemento=0.60, 0.65, 0.70 de la siguiente manera:
   - Haciendo variar la cantidad de agua se encuentra aquella cantidad que de un asentamiento de 5".
   - De esta manera se encontró que para la relaciones de agua/cemento: 0.60, 0.65, 0.70 las cantidades de agua son: 215, 214, 213 lts. respectivamente.
   - Con el porcentaje de arena y cantidad de agua optimo se procede a realizar el diseño de mezcla final para cada relación de agua/cemento .

3. Para la dosificación de fibra de acero se utilizó el siguiente criterio:

$$\boxed{\text{Dosificación} = \frac{CxT}{S}}$$

Donde:

C = Cantidad de fibra de acero (kg/m^3 de concreto)

T = Tanda de la mezcla (kg)

S = Suma de los pesos húmedos de Cemento+Agua+Ag.Fino+Ag.Grueso (kg)

DISEÑO DE MEZCLA PRELIMINAR

DISEÑO 1:

%Arena	0.47
A/C	0.65
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	200
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P.U.Sec	Proporciones P.U.Hum	Tanda de 50 kg
Cemento	307.69	0.099	307.69	1.00	1.00	6.45
Agua	200.00	0.200	210.43	0.65	0.68	4.41
Arena	870.11	0.326	883.07	2.83	2.87	18.51
Piedra	981.18	0.356	983.80	3.19	3.20	20.62
Sum. Total	2358.98	0.980	2385.00	7.67	7.75	50.0
%Aire de Diseño =		1.50%		Asentamiento :		3 7/8"
S. Parcial (Cem+Agua+Aire) =		0.314				
Sum. Total =		1.00				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.686

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.47

Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.330
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.357
Suma = 0.686

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 870.11
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 981.18

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 883.07
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 983.80

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.440
Agregado Grueso = Psx(%w-%Abs)/100 = -1.992
Corrección = -10.432

Agua Corregida = Agua - Corrección = 210.43

DISEÑO DE MEZCLA PRELIMINAR

DISEÑO 2:

%Arena	0.47
A/C	0.65
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	210
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P.U.Seco	Proporciones P. U. Hum	Tanda de 50 kg
Cemento	323.08	0.104	323.08	1.00	1.00	6.82
Agua	210.00	0.210	220.20	0.65	0.68	4.65
Arena	851.18	0.319	863.86	2.63	2.67	18.23
Piedra	959.84	0.348	962.40	2.97	2.98	20.31
Sum. Total	2344.10	0.980	2369.55	7.26	7.33	50.0
%Aire de Diseño =	1.50%			Asentamiento :		5 1/4"
S. Parcial (Cem+Agua+Aire) =	0.329					
Sum. Total =	1.00					

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.671

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.47
Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.322
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.349
Suma = 0.671

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 851.18
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 959.84

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 863.86
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 962.40

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.256
Agregado Grueso = Psx(%w-%Abs)/100 = -1.948
Corrección = -10.205

Agua Corregida = Agua - Corrección = 220.20

DISEÑO DE MEZCLA PRELIMINAR

DISEÑO 3:

%Arena	0.47
A/C	0.65
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	220
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Dosificación por m^3 de Concreto				Tanda de Prueba		
Material	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P.U.Seco	Proporciones P.U.Hum	Tanda de 50 kg
Cemento	338.46	0.108	338.46	1.00	1.00	7.19
Agua	220.00	0.220	229.98	0.65	0.68	4.88
Arena	832.25	0.312	844.65	2.46	2.50	17.94
Piedra	938.50	0.340	941.00	2.77	2.78	19.99
Sum. Total	2329.21	0.980	2354.10	6.88	6.96	50.0
%Aire de Diseño =		1.50%		Asentamiento :		7"
S. Parcial (Cem+Agua+Aire) =		0.343				
Sum. Total =		1.00				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.657

Peso seco = Peso Especifico x Volumen

Peso seco Agreg. Fino = 2640 xVa

Peso seco Agreg. Grueso = 2750 xVp

rf = 0.47

Entonces:

V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.315

V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.341

Suma = 0.657

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 832.25

Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 938.50

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 844.65

Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 941.00

Correccion de Agua :

Agregado Fino = Asx(%w-%Abs)/100 = -8.073

Agregado Grueso = Psx(%w-%Abs)/100 = -1.905

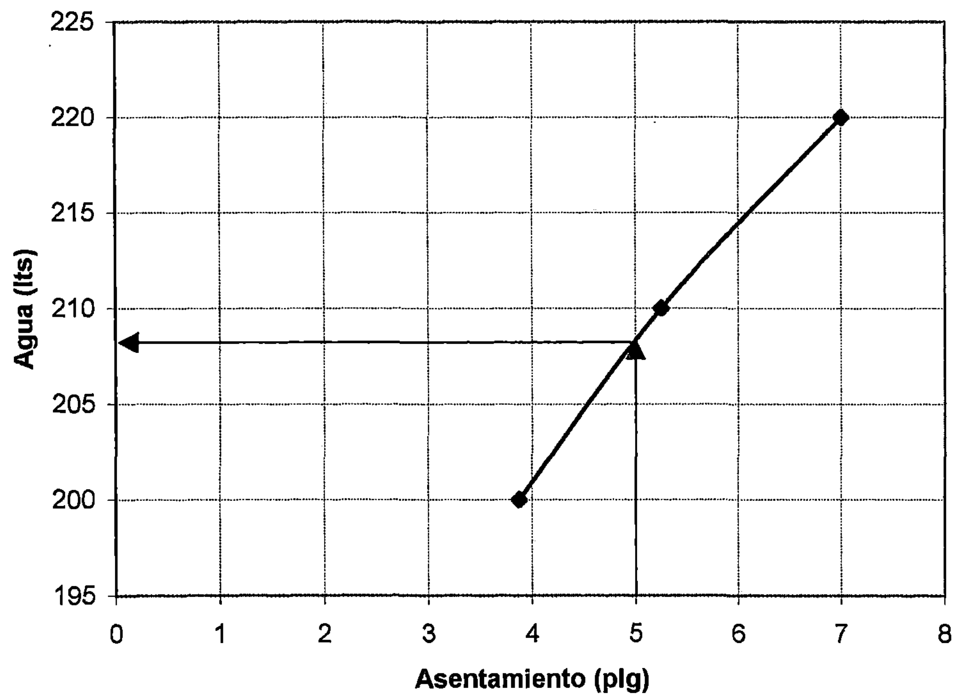
Corrección = -9.978

Agua Corregida = Agua - Corrección = 229.98

Asent. (plg)	Agua (lts)
3 7/8	200
5 1/4	210
7	220

Asent. (plg)	Agua (lts)
5	208

GRAFICA 1 ASENTAMIENTO VS AGUA
% Arena = 47 a/c = 0.65



DISEÑO DE MEZCLA PRELIMINAR

DISEÑO 4:

%Arena	0.50
A/C	0.65
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	200
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Dosificación por m^3 de Concreto				Tanda de Prueba		
Material	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P.U.Seco	P.U.Hum	Tanda de 50 kg
Cemento	307.69	0.099	307.69	1.00	1.00	6.45
Agua	200.00	0.200	210.84	0.65	0.69	4.42
Arena	924.51	0.346	938.29	3.00	3.05	19.68
Piedra	924.51	0.335	926.98	3.00	3.01	19.44
Sum. Total	2356.72	0.980	2383.81	7.66	7.75	50.0
%Aire de Diseño =		1.50%		Asentamiento :		3 1/4"
S. Parcial (Cem+Agua+Aire) =		0.314				
Sum. Total =		0.99				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.686

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.5

Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.350
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.336
Suma = 0.686

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 924.51
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 924.51

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 938.29
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 926.98

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.968
Agregado Grueso = Psx(%w-%Abs)/100 = -1.877
Corrección = -10.845

Agua Corregida = Agua - Corrección = 210.84

DISEÑO DE MEZCLA PRELIMINAR

DISEÑO 5:

%Arena	0.50
A/C	0.65
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	210
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Dosificación por m^3 de Concreto				Tanda de Prueba		
Material	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P.U.Seco	Proporciones P.U.Hum	Tanda de 50 kg
Cemento	323.08	0.104	323.08	1.00	1.00	6.82
Agua	210.00	0.210	220.61	0.65	0.68	4.66
Arena	904.40	0.339	917.88	2.80	2.84	19.38
Piedra	904.40	0.328	906.82	2.80	2.81	19.14
Sum. Total	2341.88	0.980	2368.38	7.25	7.33	50.0
%Aire de Diseño =		1.50%		Asentamiento :		4 1/2"
S. Parcial (Cem+Agua+Aire) =		0.329				
Sum. Total =		0.99				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.671

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.5

Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.343
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.329
Suma = 0.671

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 904.40
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 904.40

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 917.88
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 906.82

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.773
Agregado Grueso = Psx(%w-%Abs)/100 = -1.836
Corrección = -10.609

Agua Corregida = Agua - Corrección = 220.61

DISEÑO DE MEZCLA PRELIMINAR

DISEÑO 6:

%Arena	0.50
A/C	0.65
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	220
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P.U.Seco	Proporciones P.U.Hum	Tanda de 50 kg
Cemento	338.46	0.108	338.46	1.00	1.00	7.19
Agua	220.00	0.220	230.37	0.65	0.68	4.90
Arena	884.29	0.331	897.47	2.61	2.65	19.07
Piedra	884.29	0.320	886.65	2.61	2.62	18.84
Sum. Total	2327.04	0.980	2352.95	6.88	6.95	50.0
%Aire de Diseño =	1.50%			Asentamiento :		7"
S. Parcial (Cem+Agua+Aire) =	0.343					
Sum. Total =	1.00					

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.657

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.5

Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.335
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.322
Suma = 0.657

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 884.29
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 884.29

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 897.47
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 886.65

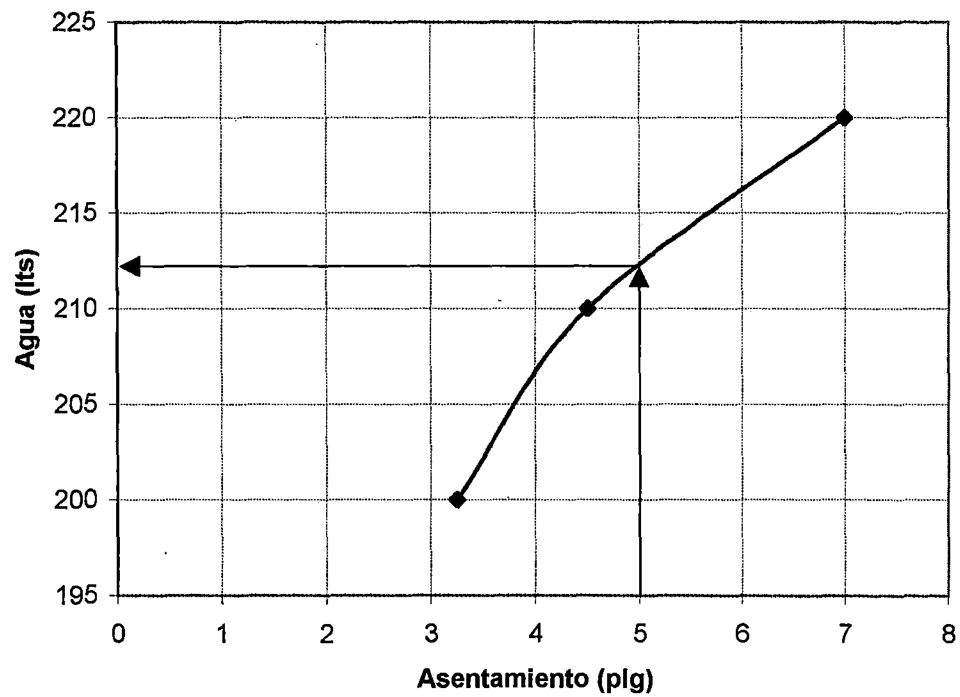
Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.578
Agregado Grueso = Psx(%w-%Abs)/100 = -1.795
Corrección = -10.373

Agua Corregida = Agua - Corrección = 230.37

GRAFICA 2 ASENTAMIENTO VS AGUA
% Arena = 50 a/c = 0.65

Asent. (plg)	Agua (lts)
3 1/4	200
4 1/2	210
7	220

Asent. (plg)	Agua (lts)
5	215



DISEÑO DE MEZCLA PRELIMINAR

DISEÑO 7:

%Arena	0.53
A/C	0.65
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	200
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P.U.Seco	Proporciones P.U.Hum	Tanda de 50 kg
Cemento	307.69	0.099	307.69	1.00	1.00	6.46
Agua	200.00	0.200	211.26	0.65	0.69	4.43
Arena	978.78	0.367	993.37	3.18	3.23	20.85
Piedra	867.98	0.314	870.30	2.82	2.83	18.26
Sum. Total	2354.46	0.980	2382.61	7.65	7.74	50.0
%Aire de Diseño =		1.50%		Asentamiento :		2 1/8"
S. Parcial (Cem+Agua+Aire) =		0.314				
Sum. Total =		0.99				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.686

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.53

Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.371
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.316
Suma = 0.686

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 978.78
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 867.98

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 993.37
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 870.30

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -9.494
Agregado Grueso = Psx(%w-%Abs)/100 = -1.762
Corrección = -11.256

Agua Corregida = Agua - Corrección = 211.26

DISEÑO DE MEZCLA PRELIMINAR

DISEÑO 8:

%Arena	0.53
A/C	0.65
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	210
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco	Vol. Abs	Peso Humedo	Proporciones		Tanda de
	(kg/m3)	(m3)	(kg/m3)	P.U.Seco	P.U.Hum	50 kg
Cemento	323.08	0.104	323.08	1.00	1.00	6.82
Agua	210.00	0.210	221.01	0.65	0.68	4.67
Arena	957.49	0.359	971.76	2.96	3.01	20.53
Piedra	849.10	0.308	851.36	2.63	2.64	17.98
Sum. Total	2339.67	0.980	2367.21	7.24	7.33	50.0
%Aire de Diseño =	1.50%			Asentamiento :		3 7/8"
S. Parcial (Cem+Agua+Aire) =	0.329					
Sum. Total =	0.99					

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.671

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.53
Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.363
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.309
Suma = 0.671

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 957.49
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 849.10

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 971.76
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 851.36

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -9.288
Agregado Grueso = Psx(%w-%Abs)/100 = -1.724
Corrección = -11.011

Agua Corregida = Agua - Corrección = 221.01

DISEÑO DE MEZCLA PRELIMINAR

DISEÑO 9:

%Arena	0.53
A/C	0.65
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	220
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P U.Seco	P.U.Hum	Tanda de 50 kg
Cemento	338.46	0.108	338.46	1.00	1.00	7.20
Agua	220.00	0.220	230.77	0.65	0.68	4.91
Arena	936.20	0.351	950.15	2.77	2.81	20.20
Piedra	830.22	0.301	832.43	2.45	2.46	17.70
Sum. Total	2324.88	0.980	2351.81	6.87	6.95	50.0
%Aire de Diseño =	1.50%			Asentamiento :		6 1/2"
S. Parcial (Cem+Agua+Aire) =	0.343					
Sum. Total =	0.99					

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.657

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.53
Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.355
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.302
Suma = 0.657

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 936.20
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 830.22

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 950.15
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 832.43

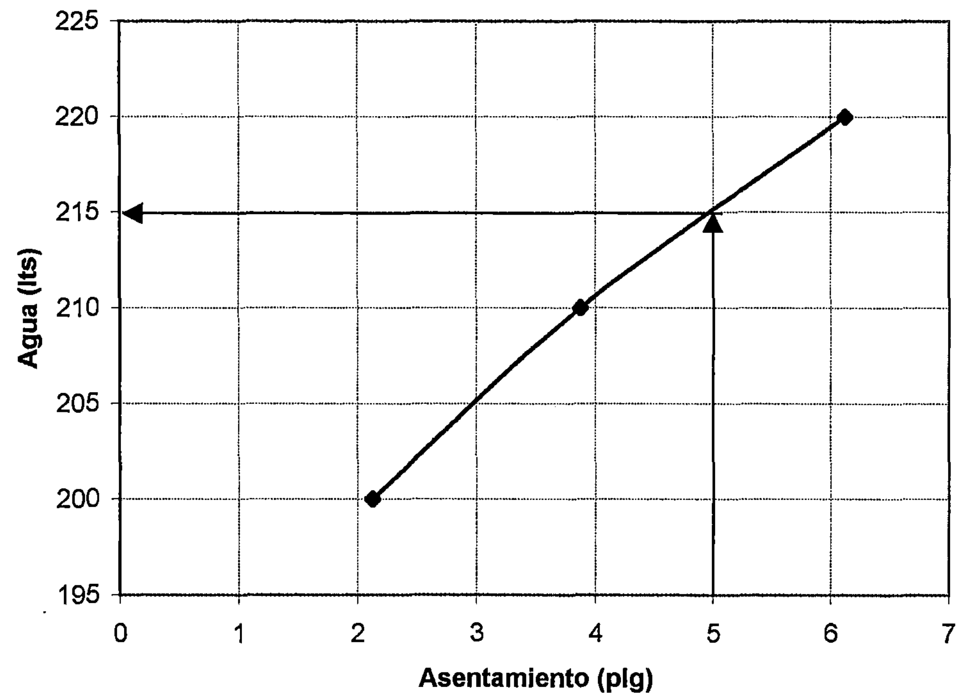
Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -9.081
Agregado Grueso = Psx(%w-%Abs)/100 = -1.685
Corrección = -10.766

Agua Corregida = Agua - Corrección = 230.77

GRAFICA 3 ASENTAMIENTO VS AGUA
% Arena = 53 a/c = 0.65

Asent. (plg)	Agua (lts)
2 1/8	200
3 7/8	210
6 1/8	220

Asent. (plg)	Agua (lts)
5	213



DISEÑO DE MEZCLA PARA F´c MAX

DISEÑO 1:

%Arena	0.47
A/C	0.65
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	208
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P.U.Seco	Proporciones P.U.Hum	Tanda de 50 kg
Cemento	320.00	0.103	320.00	1.00	1.00	6.74
Agua	208.00	0.208	218.25	0.65	0.68	4.60
Arena	854.97	0.320	867.70	2.67	2.71	18.29
Piedra	964.11	0.349	966.68	3.01	3.02	20.37
Sum. Total	2347.08	0.980	2372.64	7.33	7.41	50.0
%Aire de Diseño =		1.50%		Asentamiento :		4 1/2"
S. Parcial (Cem+Agua+Aire) =		0.326				
Sum. Total =		1.00				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.674

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.47

Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.324
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.351
Suma = 0.674

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 854.97
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 964.11

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 867.70
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 966.68

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.293
Agregado Grueso = Psx(%w-%Abs)/100 = -1.957
Corrección = -10.250

Agua Corregida = Agua - Corrección = 218.25

DISEÑO DE MEZCLA PARA F´c MAX

DISEÑO 2:

%Arena	0.50
A/C	0.65
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	213
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P.U.Seco	Proporciones P.U.Hum	Tanda de 50 kg
Cemento	327.69	0.105	327.69	1.00	1.00	6.93
Agua	213.00	0.213	223.54	0.65	0.68	4.73
Arena	898.37	0.336	911.75	2.74	2.78	19.29
Piedra	898.37	0.325	900.77	2.74	2.75	19.05
Sum. Total	2337.43	0.980	2363.75	7.13	7.21	50.0
%Aire de Diseño =		1.50%		Asentamiento :		5 1/2"
S. Parcial (Cem+Agua+Aire) =		0.333				
Sum. Total =		0.99				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.667

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.5
Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.340
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.327
Suma = 0.667

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 898.37
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 898.37

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 911.75
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 900.77

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.714
Agregado Grueso = Psx(%w-%Abs)/100 = -1.824
Corrección = -10.538

Agua Corregida = Agua - Corrección = 223.54

DISEÑO DE MEZCLA PARA F´c MAX

DISEÑO 3:

%Arena	0.53
A/C	0.65
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	215
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Dosificación por m^3 de Concreto				Tanda de Prueba		
Material	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P.U.Seco	Proporciones P.U.Hum	Tanda de 50 kg
Cemento	330.77	0.106	330.77	1.00	1.00	7.01
Agua	215.00	0.215	225.89	0.65	0.68	4.79
Arena	946.85	0.355	960.96	2.86	2.91	20.36
Piedra	839.66	0.304	841.90	2.54	2.55	17.84
Sum. Total	2332.27	0.980	2359.51	7.05	7.13	50.0
%Aire de Diseño =		1.50%		Asentamiento :		5 1/2"
S. Parcial (Cem+Agua+Aire) =		0.336				
Sum. Total =		0.99				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.664

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.53

Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.359
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.305
Suma = 0.664

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 946.85
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 839.66

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 960.96
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 841.90

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -9.184
Agregado Grueso = Psx(%w-%Abs)/100 = -1.705
Corrección = -10.889

Agua Corregida = Agua - Corrección = 225.89

DISEÑO DE MEZCLA PARA EL AGUA OPTIMA A/C=0.60

DISEÑO 1:

%Arena	0.51
A/C	0.60
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	210
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco	Vol. Abs	Peso Humedo	Proporciones		Tanda de
	(kg/m3)	(m3)	(kg/m3)	P.U.Seco	P.U.Hum	50 kg
Cemento	350.00	0.112	350.00	1.00	1.00	7.38
Agua	210.00	0.210	220.60	0.60	0.63	4.65
Arena	910.26	0.341	923.83	2.60	2.64	19.48
Piedra	874.57	0.317	876.90	2.50	2.51	18.49
Sum. Total	2344.83	0.980	2371.33	6.70	6.78	50.0
%Aire de Diseño =	1.50%			Asentamiento :		4 1/2"
S. Parcial (Cem+Agua+Aire) =	0.337					
Sum. Total =	0.99					

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.663

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.51

Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.345
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.318
Suma = 0.663

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 910.26
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 874.57

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 923.83
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 876.90

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.830
Agregado Grueso = Psx(%w-%Abs)/100 = -1.775
Corrección = -10.605

Agua Corregida = Agua - Corrección = 220.60

DISEÑO DE MEZCLA PARA EL AGUA OPTIMA A/C=0.60

DISEÑO 2:

%Arena	0.51
A/C	0.60
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	215
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P.U.Seco	Proporciones P.U.Hum	Tanda de 50 kg
Cemento	358.33	0.115	358.33	1.00	1.00	7.58
Agua	215.00	0.215	225.48	0.60	0.63	4.77
Arena	899.73	0.337	913.13	2.51	2.55	19.32
Piedra	864.44	0.313	866.75	2.41	2.42	18.33
Sum. Total	2337.51	0.980	2363.70	6.52	6.60	50.0
%Aire de Diseño =		1.50%		Asentamiento :		5"
S. Parcial (Cem+Agua+Aire) =		0.345				
Sum. Total =		1.00				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.655

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.51
Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.341
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.314
Suma = 0.655

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 899.73
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 864.44

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 913.13
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 866.75

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.727
Agregado Grueso = Psx(%w-%Abs)/100 = -1.755
Corrección = -10.482

Agua Corregida = Agua - Corrección = 225.48

DISEÑO DE MEZCLA PARA EL AGUA OPTIMA A/C=0.60

DISEÑO 3:

%Arena	0.51
A/C	0.60
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	220
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P.U.Seco	Proporciones P.U.Hum	Tanda de 50 kg
Cemento	366.67	0.118	366.67	1.00	1.00	7.78
Agua	220.00	0.220	230.36	0.60	0.63	4.89
Arena	889.19	0.333	902.44	2.43	2.46	19.15
Piedra	854.32	0.310	856.60	2.33	2.34	18.18
Sum. Total	2330.18	0.980	2356.07	6.36	6.43	50.0
%Aire de Diseño =		1.50%		Asentamiento :		6 7/8"
S. Parcial (Cem+Agua+Aire) =		0.353				
Sum. Total =		1.00				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.647

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.51

Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.337
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.311
Suma = 0.647

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 889.19
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 854.32

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 902.44
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 856.60

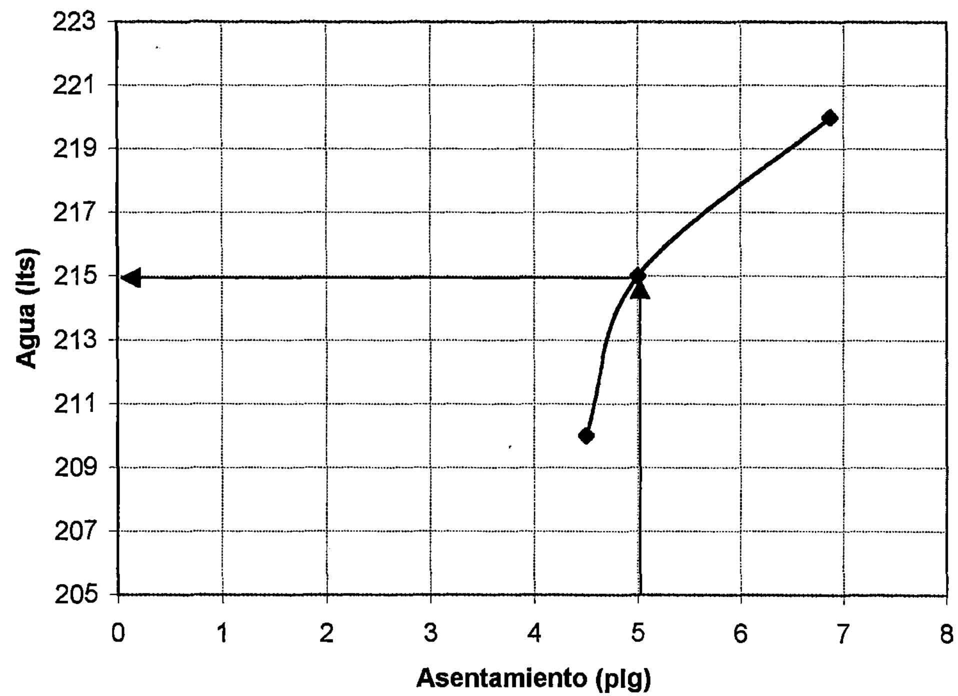
Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.625
Agregado Grueso = Psx(%w-%Abs)/100 = -1.734
Corrección = -10.359

Agua Corregida = Agua - Corrección = 230.36

GRAFICA 4 ASENTAMIENTO VS AGUA
% Arena = 51 a/c = 0.60

Asent. (plg)	Agua (lts)
4 1/2	210
5	215
6 7/8	220

Asent. (plg)	Agua (lts)
5	215



DISEÑO DE MEZCLA PARA EL AGUA OPTIMA A/C=0.65

DISEÑO 1:

%Arena	0.51
A/C	0.65
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	210
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P.U.Seco	Proporciones P.U.Hum	Tanda de 50 kg
Cemento	323.08	0.104	323.08	1.00	1.00	6.82
Agua	210.00	0.210	220.74	0.65	0.68	4.66
Arena	922.11	0.345	935.85	2.85	2.90	19.76
Piedra	885.95	0.321	888.32	2.74	2.75	18.76
Sum. Total	2341.14	0.980	2367.99	7.25	7.33	50.0
%Aire de Diseño =	1.50%			Asentamiento :		4"
S. Parcial (Cem+Agua+Aire) =	0.329					
Sum. Total =	0.99					

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.671

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.51
Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.349
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.322
Suma = 0.671

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 922.11
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 885.95

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 935.85
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 888.32

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.944
Agregado Grueso = Psx(%w-%Abs)/100 = -1.798
Corrección = -10.743

Agua Corregida = Agua - Corrección = 220.74

DISEÑO DE MEZCLA PARA EL AGUA OPTIMA A/C=0.65

DISEÑO 2:

%Arena	0.51
A/C	0.65
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	213
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Dosificación por m^3 de Concreto				Tanda de Prueba		
Material	Peso Seco	Vol. Abs	Peso Humedo	Proporciones		Tanda de
	(kg/m3)	(m3)	(kg/m3)	P.U.Seco	P.U.Hum	50 kg
Cemento	327.69	0.105	327.69	1.00	1.00	6.93
Agua	213.00	0.213	223.67	0.65	0.68	4.73
Arena	915.96	0.343	929.61	2.80	2.84	19.67
Piedra	880.04	0.319	882.39	2.69	2.69	18.67
Sum. Total	2336.70	0.980	2363.36	7.13	7.21	50.0
%Aire de Diseño =		1.50%		Asentamiento :		4 1/2"
S. Parcial (Cem+Agua+Aire) =		0.333				
Sum. Total =		0.99				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.667

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.51
Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.347
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.320
Suma = 0.667

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 915.96
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 880.04

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 929.61
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 882.39

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.885
Agregado Grueso = Psx(%w-%Abs)/100 = -1.786
Corrección = -10.671

Agua Corregida = Agua - Corrección = 223.67

DISEÑO DE MEZCLA PARA EL AGUA OPTIMA A/C=0.65

DISEÑO 3:

%Arena	0.51
A/C	0.65
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	215
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones		Tanda de 50 kg
				P. U. Seco	. U. Humed	
Cemento	330.77	0.106	330.77	1.00	1.00	7.01
Agua	215.00	0.215	225.62	0.65	0.68	4.78
Arena	911.86	0.342	925.45	2.76	2.80	19.60
Piedra	876.10	0.317	878.44	2.65	2.66	18.61
Sum. Total	2333.73	0.980	2360.28	7.06	7.14	50.0
%Aire de Diseño =		1.50%		Asentamiento :		5 1/2"
S. Parcial (Cem+Agua+Aire) =		0.336				
Sum. Total =		0.99				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.664

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.51
Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.345
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.319
Suma = 0.664

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 911.86
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 876.10

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 925.45
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 878.44

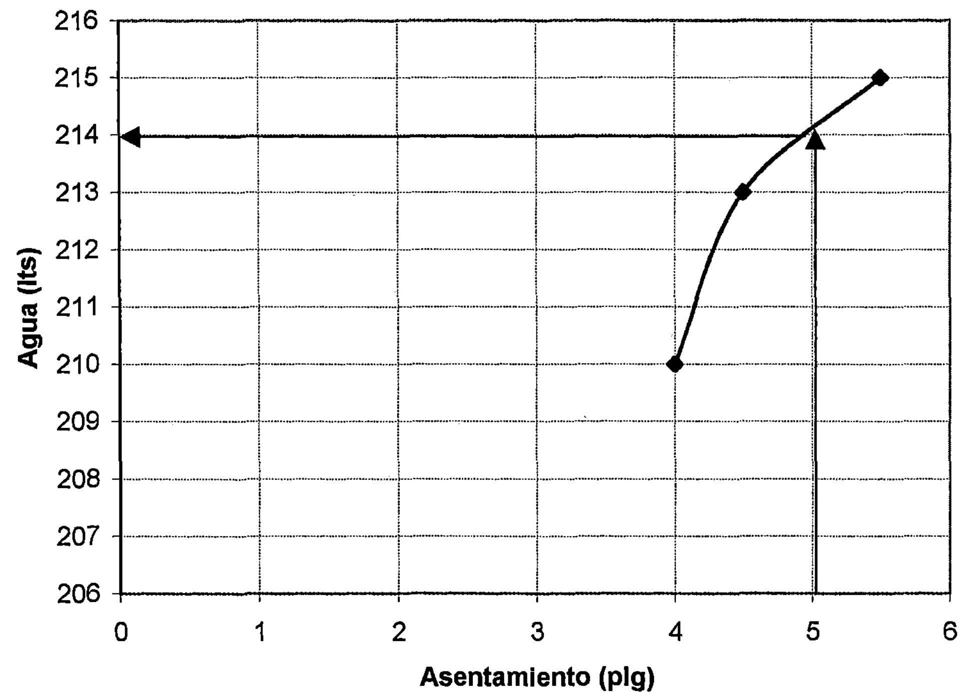
Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.845
Agregado Grueso = Psx(%w-%Abs)/100 = -1.778
Corrección = -10.624

Agua Corregida = Agua - Corrección = 225.62

GRAFICA 5 ASENTAMIENTO VS AGUA
% Arena = 51 a/c = 0.65

Asent. (plg)	Agua (lts)
4	210
4 1/2	213
5 1/2	215

Asent. (plg)	Agua (lts)
5	215



DISEÑO DE MEZCLA PARA EL AGUA OPTIMA A/C=0.70

DISEÑO 1:

%Arena	0.51
A/C	0.70
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	200
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P. U. Seco	Proporciones . U. Humed	Tanda de 50 kg
Cemento	285.71	0.092	285.71	1.00	1.00	6.00
Agua	200.00	0.200	211.09	0.70	0.74	4.43
Arena	952.29	0.357	966.48	3.33	3.38	20.30
Piedra	914.95	0.332	917.39	3.20	3.21	19.27
Sum. Total	2352.95	0.980	2380.68	8.24	8.33	50.0
%Aire de Diseño =		1.50%		Asentamiento :		2"
S. Parcial (Cem+Agua+Aire) =		0.307				
Sum. Total =		0.99				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.693

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.51
Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.361
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.333
Suma = 0.693

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 952.29
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 914.95

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 966.48
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 917.39

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -9.237
Agregado Grueso = Psx(%w-%Abs)/100 = -1.857
Corrección = -11.095

Agua Corregida = Agua - Corrección = 211.09

DISEÑO DE MEZCLA PARA EL AGUA OPTIMA A/C=0.70

DISEÑO 2:

%Arena	0.51
A/C	0.70
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	210
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P. U. Seco	Proporciones . U. Humed	Tanda de 50 kg
Cemento	300.00	0.096	300.00	1.00	1.00	6.34
Agua	210.00	0.210	220.86	0.70	0.74	4.67
Arena	932.27	0.349	946.16	3.11	3.15	20.00
Piedra	895.71	0.325	898.10	2.99	2.99	18.99
Sum. Total	2337.98	0.980	2365.13	7.79	7.88	50.0
%Aire de Diseño =	1.50%			Asentamiento :		3 3/4"
S. Parcial (Cem+Agua+Aire) =	0.321					
Sum. Total =	0.99					

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.679

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.51
Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.353
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.326
Suma = 0.679

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 932.27
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 895.71

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 946.16
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 898.10

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -9.043
Agregado Grueso = Psx(%w-%Abs)/100 = -1.818
Corrección = -10.861

Agua Corregida = Agua - Corrección = 220.86

DISEÑO DE MEZCLA PARA EL AGUA OPTIMA A/C=0.70

DISEÑO 2:

%Arena	0.51
A/C	0.70
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	215
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P. U. Seco	Proporciones . U. Humed	Tanda de 50 kg
Cemento	307.14	0.098	307.14	1.00	1.00	6.51
Agua	215.00	0.215	225.74	0.70	0.73	4.79
Arena	922.26	0.345	936.00	3.00	3.05	19.85
Piedra	886.09	0.321	888.46	2.88	2.89	18.84
Sum. Total	2330.50	0.980	2357.35	7.59	7.68	50.0
%Aire de Diseño =		1.50%		Asentamiento :		6 1/2"
S. Parcial (Cem+Agua+Aire) =		0.328				
Sum. Total =		0.99				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.672

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.51
Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.349
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.322
Suma = 0.672

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 922.26
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 886.09

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 936.00
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 888.46

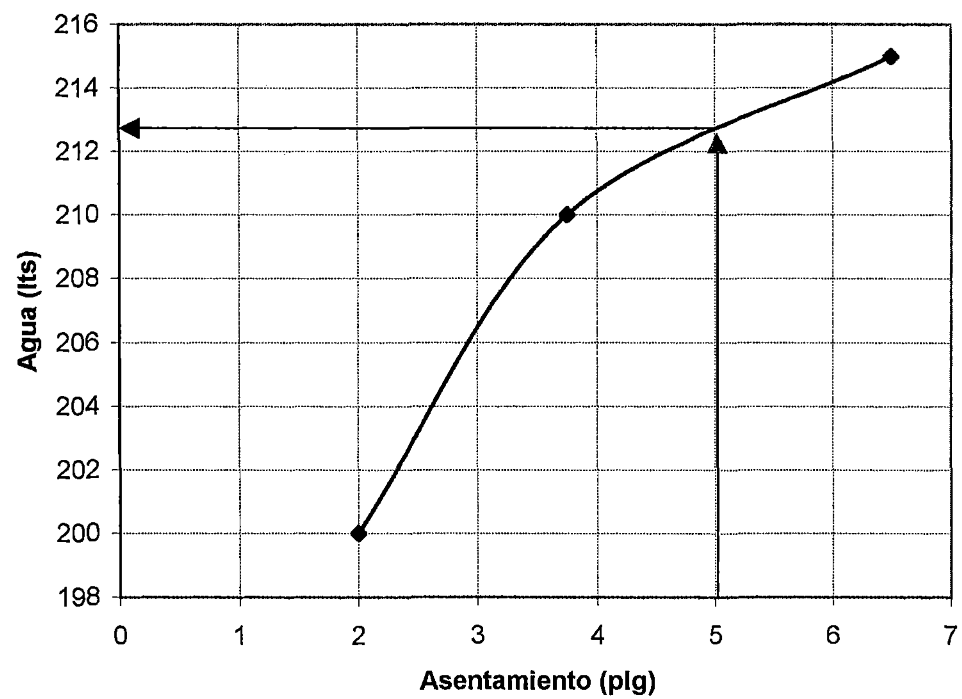
Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.946
Agregado Grueso = Psx(%w-%Abs)/100 = -1.799
Corrección = -10.745

Agua Corregida = Agua - Corrección = 225.74

GRAFICA 6 ASENTAMIENTO VS AGUA
% Arena = 51 a/c = 0.70

Asent. (plg)	Agua (lts)
2	200
3 3/4	210
6 1/2	215

Asent. (plg)	Agua (lts)
5	213



DISEÑO DE MEZCLA FINALPARA A/C=0.60

DISEÑO 1:

%Arena	0.51
A/C	0.60
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	215
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Dosificación por m^3 de Concreto				Tanda de Prueba		
Material	Peso Seco	Vol. Abs	Peso Humedo	Proporciones		Tanda de
	(kg/m3)	(m3)	(kg/m3)	P. U. Seco	. U. Humed	50 kg
Cemento	358.33	0.115	358.33	1.00	1.00	7.58
Agua	215.00	0.215	225.48	0.60	0.63	4.77
Arena	899.73	0.337	913.13	2.51	2.55	19.32
Piedra	864.44	0.313	866.75	2.41	2.42	18.33
Sum. Total	2337.51	0.980	2363.70	6.52	6.60	50.0
%Aire de Diseño =		1.50%		Asentamiento :		4 1/2"
S. Parcial (Cem+Agua+Aire) =		0.345				
Sum. Total =		1.00				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.655

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.51
Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.341
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.314
Suma = 0.655

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 899.73
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 864.44

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 913.13
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 866.75

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.727
Agregado Grueso = Psx(%w-%Abs)/100 = -1.755
Corrección = -10.482

Agua Corregida = Agua - Corrección = 225.48

DISEÑO DE MEZCLA FINALPARA A/C=0.65

DISEÑO 1:

%Arena	0.51
A/C	0.65
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	214
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Dosificación por m^3 de Concreto				Tanda de Prueba		
Material	Peso Seco	Vol. Abs	Peso Humedo	Proporciones		Tanda de
	(kg/m3)	(m3)	(kg/m3)	P. U. Seco	. U. Humed	50 kg
Cemento	329.23	0.106	329.23	1.00	1.00	6.97
Agua	214.00	0.214	224.65	0.65	0.68	4.76
Arena	913.91	0.342	927.53	2.78	2.82	19.64
Piedra	878.07	0.318	880.42	2.67	2.67	18.64
Sum. Total	2335.21	0.980	2361.82	7.09	7.17	50.0
%Aire de Diseño =		1.50%		Asentamiento :		4 1/2"
S. Parcial (Cem+Agua+Aire) =		0.335				
Sum. Total =		0.99				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.665

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.51

Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.346
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.319
Suma = 0.665

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 913.91
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 878.07

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 927.53
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 880.42

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.865
Agregado Grueso = Psx(%w-%Abs)/100 = -1.782
Corrección = -10.647

Agua Corregida = Agua - Corrección = 224.65

DISEÑO DE MEZCLA FINALPARA A/C=0.70

DISEÑO 1:

%Arena	0.51
A/C	0.70
Asent.	41/2"-51/2"
T.N.Max	1"
Agua	213
% Aire	1.50%

DATOS :

Descripción	Ag. Fino	Ag. Grueso	Cemento
Peso Especifico (kg/m^3)	2640	2750	3120
Cont. de Humedad (%)	1.490	0.267	
Porc. de Absorción (%)	2.460	0.470	

Material	Dosificación por m^3 de Concreto			Tanda de Prueba		
	Peso Seco (kg/m3)	Vol. Abs (m3)	Peso Humedo (kg/m3)	Proporciones P. U. Seco	Proporciones . U. Humed	Tanda de 50 kg
Cemento	304.29	0.098	304.29	1.00	1.00	6.45
Agua	213.00	0.213	223.79	0.70	0.74	4.74
Arena	926.26	0.347	940.07	3.04	3.09	19.91
Piedra	889.94	0.322	892.32	2.92	2.93	18.90
Sum. Total	2333.49	0.980	2360.46	7.67	7.76	50.0
%Aire de Diseño =		1.50%		Asentamiento :		5"
S. Parcial (Cem+Agua+Aire) =		0.326				
Sum. Total =		0.99				

1) DISEÑO SECO:

Vol.Tot = Vol.Fino + Grueso = 0.674

Peso seco = Peso Especifico x Volumen
Peso seco Agreg. Fino = 2640 xVa
Peso seco Agreg. Grueso = 2750 xVp
rf = 0.51
Entonces:
V Ag. Fino = rfxVTxPepd/(Pear + rf(Pepd-Pear)) = 0.351
V Ag. Grueso = Vol. Total - Vol. Agregado. Fino = 0.324
Suma = 0.674

Peso seco Agreg. Fino = 2640 xV. Ag.Fino = 926.26
Peso seco Agreg. Grueso = 2750 xV. Ag.Grueso = 889.94

2) DISEÑO HUMEDO:

Peso Humedo Agreg. Fino = Asx(1+%w/100) = 940.07
Peso Humedo Agreg. Grueso = Psx(1+%w/100) = 892.32

Correccion de Agua :
Agregado Fino = Asx(%w-%Abs)/100 = -8.985
Agregado Grueso = Psx(%w-%Abs)/100 = -1.807
Corrección = -10.791

Agua Corregida = Agua - Corrección = 223.79

ANEXO C: CUADROS DE ENSAYOS EN ESTADO FRESCO Y ENDURECIDO

I. ENSAYOS EN EL ESTADO FRESCO DELCONCRETO PATRON

1.-ENSAYO DE ASENTAMIENTO

Relación a/c	Asentamiento
0.60	4 1/2"
0.65	4 1/2"
0.70	5"

2.-ENSAYO DE FLUIDEZ

Relación	Diámetro (cm)							Fluidez
a/c	D1	D2	D3	D4	D5	D6	Prom	(%)
0.60	52.00	52.00	52.00	53.50	52.00	52.00	52.25	105.71
0.65	54.00	54.00	53.00	53.00	54.30	53.00	53.55	110.83
0.70	55.00	54.00	54.00	54.00	55.00	54.00	54.33	113.91

3.-ENSAYO DE PESO UNITARIO COMPACTADO

Relación a/c	Vbalde (m^3)	Wbalde (Kg)	Wbalde+ Wmezcla	Wmezcla (Kg)	P.U.C. (Kg/m^3)
0.60	0.01	8.60	42.80	34.20	2442.86
0.65	0.01	8.60	42.90	34.30	2450.00
0.70	0.01	8.60	42.90	34.30	2450.00

4.-ENSAYO DE TIEMPO DE FRAGUADO

Relación a/c	Diámetro	Sección Plg^2	Tiempo Hr	Fuerza Lbs	Resist. (Lbs/plg^2)	Fraguado Iinicial	Fraguado Final
0.60			0.00		0.00	3:50	5:08
	1 1/8"	0.99402	3.00	130	130.78		
	13/16"	0.51848	3.50	170	327.88		
	9/16"	0.24850	4.00	125	503.02		
	5/16"	0.07669	4.50	140	1825.53		
	4/16"	0.04908	5.00	160	3259.98		
	3/16"	0.02761	5.50	200	7243.75		
0.65			0.00		0.00	4:05	5:15
	1 1/8"	0.99402	3.00	150	150.90		
	13/16"	0.51848	3.50	150	289.31		
	9/16"	0.24850	4.20	125	503.02		
	5/16"	0.07669	4.50	140	1825.53		
	4/16"	0.04908	5.10	160	3259.98		
	3/16"	0.02761	5.50	200	7243.75		
0.70			0.00		0.00	4:20	5:20
	1 1/8"	0.99402	3.00	130	130.78		
	13/16"	0.51848	3.60	155	298.95		
	9/16"	0.24850	4.40	130	523.14		
	5/16"	0.07669	4.75	150	1955.93		
	4/16"	0.04908	5.20	160	3259.98		
	3/16"	0.02761	5.60	195	7062.66		

5.-CONTENIDO DE AIRE

Relación a/c	Contenido de Aire (%)
0.60	1.60
0.65	1.50
0.70	1.40

6.-EXUDACION

B: Cantidad de agua exudada
b: Cantidad de agua por mt^3 en Kg
R: Peso de la mezcla
W: Peso total de materiales para mt^3 de concreto

$$C = \frac{bxR}{W}$$

$$\%Exud = \frac{B}{10xC}$$

Relación a/c	Muestra	B	b	R	W	C	%Exudac	Promedio
0.60	M-1	30.0	4.77	11.82	50	1.13	2.66	2.58
	M-2	29.0	4.77	12.12	50	1.16	2.51	
0.65	M-1	32.0	4.75	12.16	50	1.16	2.77	2.77
	M-2	33.0	4.75	12.50	50	1.19	2.78	
0.70	M-1	32.0	4.74	12.47	50	1.18	2.71	2.83
	M-2	35.0	4.74	12.54	50	1.19	2.94	

II. ENSAYOS EN EL ESTADO FRESCO DEL CONCRETO CON FIBRAS

RELACIÓN AGUA/CEMENTO DE 0.60

1.-ENSAYO DE ASENTAMIENTO

Dosif. (kg/m^3)	Asentamiento
35	3 1/2"
45	3 3/4"
55	3 1/8"

2.-ENSAYO DE FLUIDEZ

Dosif. (kg/m^3)	Diámetro (cm)							Fluidez (%)
	D1	D2	D3	D4	D5	D6	Prom	
35	52.00	51.00	52.00	51.00	51.00	52.00	51.50	102.76
45	51.00	51.00	50.00	50.00	50.00	51.00	50.50	98.82
55	49.00	50.00	49.00	49.00	49.00	48.50	49.08	93.24

3.-ENSAYO DE PESO UNITARIO

Dosif. (Kg/m^3)	Vbalde (m^3)	Wbalde (Kg)	Wbalde+ Wmezcla	Wmezcla (Kg)	P.U.C. (Kg/m^3)
35	0.01	8.60	42.85	34.25	2446.43
45	0.01	8.60	42.90	34.30	2450.00
55	0.01	8.60	43.10	34.50	2464.29

4.-ENSAYO DE TIEMPO DE FRAGUADO

Dosif. (Kg/m^3)	Diámetro	Sección Plg^2	Tiempo Hr	Fuerza Lbs	Resist. (Lbs/plg^2)	Fragua Inicial	Fragua Final
35			0.00		0.00	3:55	5:12
	1 1/8"	0.99402	3.00	170	171.02		
	13/16"	0.51848	3.50	145	279.66		
	9/16"	0.24850	4.10	125	503.02		
	5/16"	0.07669	4.50	150	1955.93		
	4/16"	0.04908	5.10	175	3565.61		
	3/16"	0.02761	5.60	200	7243.75		
45			0.00		0.00	4:05	5:18
	1 1/8"	0.99402	3.20	125	125.75		
	13/16"	0.51848	3.60	170	327.88		
	9/16"	0.24850	4.20	125	503.02		
	5/16"	0.07669	4.70	165	2151.52		
	4/16"	0.04908	5.20	170	3463.73		
	3/16"	0.02761	5.70	200	7243.75		
55			0.00		0.00	4:25	5:30
	1 1/8"	0.99402	3.50	130	130.78		
	13/16"	0.51848	3.90	150	289.31		
	9/16"	0.24850	4.40	135	543.26		
	5/16"	0.07669	4.80	135	1760.33		
	4/16"	0.04908	5.40	150	3056.23		
	3/16"	0.02761	5.80	180	6519.38		

5.-CONTENIDO DE AIRE

Dosif. (Kg/m^3)	Contenido de Aire (%)
35	1.90
45	2.10
55	2.14

6.-EXUDACION

B: Cantidad de agua exudada
b: Cantidad de agua por mt^3 en Kg
R: Peso de la mezcla
W: Peso total de materiales para mt^3 de concreto

$$C = \frac{bxR}{W}$$

$$\%Exud = \frac{B}{10xC}$$

Dosif. (Kg/m^3)	Muestra	B	b	R	W	C	%Exudac	Promedio
35	E-1	28.00	4.77	11.84	50	1.13	2.48	2.50
	E-2	29.00	4.77	12.01	50	1.15	2.53	
45	E-1	29.00	4.77	12.16	50	1.16	2.50	2.42
	E-2	28.00	4.77	12.50	50	1.19	2.35	
55	E-1	28.00	4.77	12.51	50	1.19	2.35	2.30
	E-2	27.00	4.77	12.57	50	1.20	2.25	

RELACIÓN AGUA/CEMENTO DE 0.65

1.-ENSAYO DE ASENTAMIENTO

Dosif. (Kg/m^3)	Asentamiento
35	3 1/2"
45	3 1/2"
55	3 1/4"

2.-ENSAYO DE FLUIDEZ

Dosif.	Diámetro (cm)							Fluidez
(Kg/m^3)	D1	D2	D3	D4	D5	D6	Prom	(%)
35	52.00	52.00	53.00	52.00	53.00	52.00	52.33	106.04
45	51.00	51.00	52.00	52.00	52.00	52.00	51.67	103.41
55	51.00	50.00	50.00	49.00	50.00	49.00	49.83	96.19

3.-ENSAYO DE PESO UNITARIO COMPACTADO

Dosif. (Kg/m^3)	Vbalde (m^3)	Wbalde (Kg)	Wbalde+ Wmezcla	Wmezcla (Kg)	P.U.C. (Kg/m^3)
35	0.01	8.60	43.30	34.70	2478.57
45	0.01	8.60	43.40	34.80	2485.71
55	0.01	8.60	43.50	34.90	2492.86

4.-ENSAYO DE TIEMPO DE FRAGUADO

Dosif. (Kg/m^3)	Diámetro	Sección Plg^2	Tiempo Hr	Fuerza Lbs	Resist. (Lbs/plg^2)	Fragua Inicial	Fragua Final
35			0.00		0.00	4:15	5:23
	1 1/8"	0.99402	3.00	140	140.84		
	13/16"	0.51848	3.50	160	308.59		
	9/16"	0.24850	4.50	130	523.14		
	5/16"	0.07669	5.00	130	1695.14		
	4/16"	0.04908	5.30	160	3259.98		
	3/16"	0.02761	5.80	200	7243.75		
45			0.00		0.00	4:25	5:30
	1 1/8"	0.99402	3.50	145	145.87		
	13/16"	0.51848	3.80	160	308.59		
	9/16"	0.24850	4.60	130	523.14		
	5/16"	0.07669	5.00	130	1695.14		
	4/16"	0.04908	5.40	160	3259.98		
	3/16"	0.02761	5.80	200	7243.75		
55			0.00		0.00	4:35	5:35
	1 1/8"	0.99402	3.50	120	120.72		
	13/16"	0.51848	3.80	130	250.73		
	9/16"	0.24850	4.70	145	583.50		
	5/16"	0.07669	5.00	175	2281.91		
	4/16"	0.04908	5.50	185	3769.36		
	3/16"	0.02761	6.00	200	7243.75		

5.-CONTENIDO DE AIRE

Dosif. (Kg/m^3)	Contenido de Aire (%)
35	1.70
45	1.90
55	2.05

6.-EXUDACIÓN

B: Cantidad de agua exudada
b: Cantidad de agua por mt^3 en Kg
R: Peso de la mezcla
W: Peso total de materiales para mt^3 de concreto

$$C = \frac{bxR}{W}$$

$$\%Exud = \frac{B}{10xC}$$

Dosif. (Kg/m^3)	Muestra	B	b	R	W	C	%Exudac	Promedio
35	E-1	31.00	4.75	11.95	50	1.14	2.73	2.70
	E-2	31.00	4.75	12.26	50	1.16	2.66	
45	E-1	30.00	4.75	12.29	50	1.17	2.57	2.50
	E-2	29.00	4.75	12.60	50	1.20	2.42	
55	E-1	29.00	4.75	12.51	50	1.19	2.44	2.43
	E-2	28.80	4.75	12.56	50	1.19	2.41	

RELACION AGUA CEMENTO DE 0.70

1.-ENSAYO DE ASENTAMIENTO

Dosif. (Kg/m^3)	Asentamiento
35.00	3"
45.00	3 1/2"
55.00	3"

2.-ENSAYO DE FLUIDEZ

Dosif. (Kg/m^3)	Diámetro (cm)							Fluidez
	D1	D2	D3	D4	D5	D6	Prom	(%)
35.00	53.00	52.50	53.00	52.00	53.00	52.50	52.67	107.35
45.00	51.00	50.50	52.00	51.50	51.50	52.00	51.42	102.43
55.00	51.00	49.00	51.00	49.00	50.00	51.00	50.17	97.51

3.-ENSAYO DE PESO UNITARIO

Dosif. (Kg/m^3)	Vbalde (m^3)	Wbalde (Kg)	Wbalde+ Wmezcla	Wmezcla (Kg)	P.U.C. (Kg/m^3)
35.00	0.01	8.60	43.00	34.40	2457.14
45.00	0.01	8.60	43.50	34.90	2492.86
55.00	0.01	8.60	43.60	35.00	2500.00

4.-ENSAYO DE TIEMPO DE FRAGUADO

Dosif. (Kg/m^3)	Diámetro	Sección Plg^2	Tiempo Hr	Fuerza Lbs	Resist. (Lbs/plg^2)	Fragua Inicial	Fragua Final
35.00			0.00		0.00	4:25	5:35
	1 1/8"	0.99402	3.00	120	120.72		
	13/16"	0.51848	3.50	150	289.31		
	9/16"	0.24850	4.40	120	482.90		
	5/16"	0.07669	4.80	160	2086.32		
	4/16"	0.04908	5.30	180	3667.48		
	3/16"	0.02761	5.60	170	6157.19		
45.00			0.00		0.00	4:30	5:40
	1 1/8"	0.99402	3.50	160	160.96		
	13/16"	0.51848	4.00	160	308.59		
	9/16"	0.24850	4.50	130	523.14		
	5/16"	0.07669	5.00	130	1695.14		
	4/16"	0.04908	5.50	160	3259.98		
	3/16"	0.02761	6.00	190	6881.56		
55.00			0.00		0.00	4:40	5:45
	1 1/8"	0.99402	3.50	130	130.78		
	13/16"	0.51848	4.00	130	250.73		
	9/16"	0.24850	4.50	125	503.02		
	5/16"	0.07669	5.20	130	1695.14		
	4/16"	0.04908	5.60	150	3056.23		
	3/16"	0.02761	6.00	180	6519.38		

5.-CONTENIDO DE AIRE

Dosif. (Kg/m^3)	Contenido de Aire (%)
35.00	1.50
45.00	1.80
55.00	1.90

6.-EXUDACIÓN

B: Cantidad de agua exudada
b: Cantidad de agua por mt^3 en Kg
R: Peso de la mezcla
W: Peso total de materiales para mt^3 de concreto

$$C = \frac{bxR}{W}$$

$$\%Exud = \frac{B}{10xC}$$

Dosif. (Kg/m^3)	Muestra	B	b	R	W	C	%Exudac	Promedio
35.00	E-1	32.00	4.74	11.75	50	1.11	2.87	2.79
	E-2	31.00	4.74	12.04	50	1.14	2.72	
45.00	E-1	30.00	4.74	12.12	50	1.15	2.61	2.53
	E-2	29.00	4.74	12.46	50	1.18	2.46	
55.00	E-1	29.60	4.74	12.44	50	1.18	2.51	2.44
	E-2	28.10	4.74	12.50	50	1.19	2.37	

I. ENSAYO EN EL ESTADO ENDURECIDO DEL CONCRETO PATRÓN

MÓDULO DE ELASTICIDAD ESTÁTICO

1.- RELACIÓN AGUA/CEMENTO 0.60

PESO : 13.7 kg.
DIAMETRO : 15.0 cm.
AREA : 177.0 cm2
CARGA MAX: 54600 kg.

Carga (kg)	Esfuerzo (kg/cm^2)	Lect. Izq.	Lect. Der.	Prom. Lec.	Lect. Correg.	Deform. Unit. ($x10^{-4}$) cm
0	0.00	0.00	0.00	0.00	0.00	0.00
2000	11.30	0.20	0.30	0.25	0.25	0.25
4000	22.61	0.30	0.40	0.35	0.35	0.35
6000	33.91	0.40	0.50	0.45	0.45	0.45
8000	45.21	0.50	0.60	0.55	0.55	0.55
10000	56.51	0.60	0.80	0.70	0.70	0.70
12000	67.82	1.00	1.20	1.10	1.10	1.10
14000	79.12	1.30	1.40	1.35	1.35	1.35
16000	90.42	1.60	1.80	1.70	1.70	1.70
18000	101.72	2.10	2.40	2.25	2.25	2.25
20000	113.03	2.70	2.90	2.80	2.80	2.80
22000	124.33	3.50	3.50	3.50	3.50	3.50
24000	135.63	4.10	4.00	4.05	4.05	4.05
26000	146.93	4.50	4.40	4.45	4.45	4.45
28000	158.24	5.30	5.00	5.15	5.15	5.15
30000	169.54	6.10	5.70	5.90	5.90	5.90
32000	179.37	7.00	6.50	6.75	6.75	6.75
34000	190.58	7.70	7.30	7.50	7.50	7.50
36000	201.79	8.70	8.30	8.50	8.50	8.50
38000	213.00	9.80	9.10	9.45	9.45	9.45
40000	224.22	10.80	10.10	10.45	10.45	10.45
42000	235.43	12.00	11.00	11.50	11.50	11.50
44000	246.64	13.20	12.80	13.00	13.00	13.00
46000	257.85	14.70	14.90	14.80	14.80	14.80
48000	269.06	16.30	16.40	16.35	16.35	16.35
50000	280.27	19.00	19.10	19.05	19.05	19.05
52000	291.48	20.50	20.70	20.60	20.60	20.60
54000	302.69	22.50	22.80	22.65	22.65	22.65

ROTURA = Cmax/Area = 308.56 kg/cm^2

E1 = 40%ROTURA = 123.42 kg/cm^2
Eo = 39.56 kg/cm^2
D1 = 3.44 x10^-4cm
Do = 0.50 x10^-4cm

$$M.E. = \frac{(E1 - Eo)}{(D1 - Do)x10^{-4}}$$

M.E. = 2.853 x10^5 kg/cm^2

2.- RELACIÓN AGUA/CEMENTO 0.65

PESO : 13.7 kg.
DIAMETRO : 14.91 cm.
AREA : 174.6 cm2
CARGA MAX: 48000 kg.

Carga (kg)	Esfuerzo (kg/cm^2)	Lect. Izq.	Lect. Der.	Prom. Lec.	Lect. Correg.	Deform. Unit. ($x10^{-4}$) cm
0	0.0	0.00	0.10	0.05	0.00	0.00
2000	11.45	0.20	0.20	0.20	0.15	0.15
4000	22.91	0.30	0.30	0.30	0.25	0.25
6000	34.36	0.40	0.50	0.45	0.40	0.40
8000	45.82	0.60	0.70	0.65	0.60	0.60
10000	57.27	0.70	1.00	0.85	0.80	0.80
12000	68.73	0.90	1.50	1.20	1.15	1.15
14000	80.18	1.20	2.00	1.60	1.55	1.55
16000	91.64	1.40	2.70	2.05	2.00	2.00
18000	103.09	2.20	3.40	2.80	2.75	2.75
20000	114.55	2.40	3.90	3.15	3.10	3.10
22000	126.00	2.50	4.80	3.65	3.60	3.60
24000	137.46	2.60	5.70	4.15	4.10	4.10
26000	148.91	3.00	6.50	4.75	4.70	4.70
28000	160.37	3.30	7.50	5.40	5.35	5.35
30000	171.82	3.80	8.50	6.15	6.10	6.10
32000	183.28	4.10	9.50	6.80	6.75	6.75
34000	194.73	4.40	10.50	7.45	7.40	7.40
36000	206.18	4.90	11.50	8.20	8.15	8.15
38000	217.64	5.40	13.00	9.20	9.15	9.15
40000	229.09	6.00	14.50	10.25	10.20	10.20
42000	240.55	6.50	16.50	11.50	11.45	11.45
44000	252.00	7.00	19.30	13.15	13.10	13.10
46000	263.46	7.50	20.00	13.75	13.70	13.70

ROTURA = Cmax/Area = 274.91 kg/cm^2

E1 = 40% ROTURA = 109.97 kg/cm^2
Eo = 40.00 kg/cm^2
D1 = 3.09 x10^-4cm
Do = 0.50 x10^-4cm

$$M.E. = \frac{(E1 - Eo)}{(D1 - Do)x10^{-4}}$$

M.E. = 2.701 x10^5 kg/cm^2

3.- RELACIÓN AGUA/CEMENTO 0.70

PESO : 13.70 kg.
DIAMETRO : 14.92 cm.
AREA : 174.84 cm2
CARGA MAX : 41400 kg.

Carga (kg)	Esfuerzo (kg/cm^2)	Lect. Izq.	Lect. Der.	Prom. Lec.	Lect. Correg.	Deform. Unit. ($x10^{-4}$) cm
0	0.0	0.10	0.00	0.05	0.00	0.00
2000	11.44	0.30	0.40	0.35	0.30	0.30
4000	22.88	0.60	0.50	0.55	0.50	0.50
6000	34.32	0.80	0.70	0.75	0.70	0.70
8000	45.76	1.00	1.20	1.10	1.05	1.05
10000	57.20	1.40	1.60	1.50	1.45	1.45
12000	68.64	2.10	2.00	2.05	2.00	2.00
14000	80.08	2.40	2.50	2.45	2.40	2.40
16000	91.51	3.40	3.00	3.20	3.15	3.15
18000	102.95	4.00	4.00	4.00	3.95	3.95
20000	114.39	4.80	7.50	6.15	6.10	6.10
22000	125.83	5.50	9.00	7.25	7.20	7.20
24000	137.27	6.40	10.50	8.45	8.40	8.40
26000	148.71	7.00	12.00	9.50	9.45	9.45
28000	160.15	7.90	13.50	10.70	10.65	10.65
30000	171.59	8.90	15.50	12.20	12.15	12.15
32000	183.03	9.90	17.50	13.70	13.65	13.65
34000	194.47	11.40	20.00	15.70	15.65	15.65
36000	205.91	12.70	23.00	17.85	17.80	17.80
38000	217.35	14.50	27.50	21.00	20.95	20.95
40000	228.79	17.00	30.00	23.50	23.45	23.45

ROTURA = Cmax/Area = 236.79 kg/cm^2

kg/cm^2

E1 = 40%ROTURA = 94.7 kg/cm^2
Eo = 22.9 kg/cm2
D1 = 3.3 x10^-4cm
Do = 0.5 x10^-4cm

M.E. = $\dfrac{(E1 - Eo)}{(D1 - Do)x10^{-4}}$ M.E. = 2.566 x10^5 kg/cm^2

II. ENSAYO EN EL ESTADO ENDURECIDO DEL CONCRETO CON FIBRA

MÓDULO DE ELASTICIDAD ESTÁTICO

1.- RELACIÓN AGUA/CEMENTO 0.60

DOSIFICACIÓN DE LA FIBRA 35 Kg/m^3 DE CONCRETO

PESO : 13.02 kg.
DIAMETRO : 15.00 cm.
AREA : 176.72 cm2
CARGA MAX: 51800 kg.

Carga (kg)	Esfuerzo (kg/cm^2)	Lect. Izq.	Lect. Der.	Prom. Lec.	Lect. Correg.	Deform. Unit. (x10^{-4}) cm
0.00	0.00	0.10	0.15	0.13	0.00	0.00
2000	11.32	0.40	0.60	0.50	0.38	0.38
4000	22.64	0.70	1.00	0.85	0.73	0.73
6000	33.95	1.10	1.50	1.30	1.18	1.18
8000	45.27	1.50	2.00	1.75	1.63	1.63
10000	56.59	1.90	2.50	2.20	2.08	2.08
12000	67.91	2.30	3.10	2.70	2.58	2.58
14000	79.22	2.70	3.60	3.15	3.03	3.03
16000	90.54	3.10	4.00	3.55	3.43	3.43
18000	101.86	3.50	4.50	4.00	3.88	3.88
20000	113.18	3.90	4.90	4.40	4.28	4.28
22000	124.49	4.00	5.60	4.80	4.68	4.68
24000	135.81	4.80	6.70	5.75	5.63	5.63
26000	147.13	5.30	7.00	6.15	6.03	6.03
28000	158.45	5.90	8.00	6.95	6.83	6.83
30000	169.76	6.50	8.60	7.55	7.43	7.43
32000	179.37	7.00	9.40	8.20	8.08	8.08
34000	190.58	7.70	10.50	9.10	8.98	8.98
36000	201.79	8.50	12.10	10.30	10.18	10.18
38000	213.00	9.00	12.80	10.90	10.78	10.78
40000	224.22	10.90	13.50	12.20	12.08	12.08
42000	235.43	11.00	16.20	13.60	13.48	13.48
44000	246.64	13.70	17.70	15.70	15.58	15.58
46000	257.85	15.50	19.80	17.65	17.53	17.53
48000	269.06	18.00	23.00	20.50	20.38	20.38
50000	280.27	24.00	24.00	24.00	23.88	23.88

ROTURA = Cmax/Area = 293.13 kg/cm^2

E1 = 40%ROTURA = 117.3 kg/cm^2
Eo = 15.2 kg/cm^2
D1 = 4.42 x10^-4cm
Do = 0.5 x10^-4cm

$$M.E. = \frac{(E1 - Eo)}{(D1 - Do)x10^{-4}} \qquad M.E. = 2.603\ x10^{5}\ kg/cm^2$$

DOSIFICACION DE LA FIBRA 45 Kg/m³ DE CONCRETO

PESO : 13.08 kg.
DIAMETRO : 14.86 cm.
AREA : 173.43 cm2
CARGA MAX: 50500 kg.

Carga (kg)	Esfuerzo (kg/cm^2)	Lect. Izq.	Lect. Der.	Prom. Lec.	Lect. Correg.	Deform. Unit. ($x10^{-4}$) cm
0	0.0	0.00	0.00	0.00	0.00	0.00
2000	11.53	0.60	0.30	0.45	0.45	0.45
4000	23.06	1.10	0.70	0.90	0.90	0.90
6000	34.60	1.40	1.30	1.35	1.35	1.35
8000	46.13	1.80	1.70	1.75	1.75	1.75
10000	57.66	2.30	2.30	2.30	2.30	2.30
12000	69.19	2.60	2.70	2.65	2.65	2.65
14000	80.72	3.00	3.10	3.05	3.05	3.05
16000	92.26	3.50	3.40	3.45	3.45	3.45
18000	103.79	4.00	4.00	4.00	4.00	4.00
20000	115.32	4.50	4.60	4.55	4.55	4.55
22000	126.85	5.50	5.70	5.60	5.60	5.60
24000	138.38	6.00	6.60	6.30	6.30	6.30
26000	149.91	6.60	7.30	6.95	6.95	6.95
28000	161.45	7.30	8.10	7.70	7.70	7.70
30000	172.98	7.90	8.90	8.40	8.40	8.40
32000	184.51	8.50	9.60	9.05	9.05	9.05
34000	196.04	9.00	10.50	9.75	9.75	9.75
36000	207.57	9.90	11.40	10.65	10.65	10.65
38000	219.11	10.70	12.30	11.50	11.50	11.50
40000	230.64	11.60	13.40	12.50	12.50	12.50
42000	242.17	12.50	14.50	13.50	13.50	13.50
44000	253.70	13.60	15.80	14.70	14.70	14.70
46000	265.23	15.30	17.60	16.45	16.45	16.45
48000	276.77	17.00	19.50	18.25	18.25	18.25
50000	288.30	19.50	22.10	20.80	20.80	20.80
52000	299.83	23.50	24.00	23.75	23.75	23.75

ROTURA = Cmax/Area = 291.18 kg/cm^2

E1 = 40% ROTURA = 116.47 kg/cm^2
Eo = 12.62 kg/cm^2
D1 = 4.65 x10^-4cm
Do = 0.50 x10^-4cm

$$M.E. = \frac{(E1 - Eo)}{(D1 - Do)x10^{-4}}$$

M.E. = 2.502 x10^5 kg/cm^2

DOSIFICACION DE LA FIBRA 55 Kg/m³ DE CONCRETO

PESO : 13.18 kg.
DIAMETRO : 15.10 cm.
AREA : 179.08 cm2
CARGA MAX : 48600 kg.

Carga (kg)	Esfuerzo (kg/cm^2)	Lect. Izq.	Lect. Der.	Prom. Lec.	Lect. Correg.	Deform. Unit. ($x10^{-4}$) cm
0	0.0	-0.10	0.00	-0.05	0.00	0.00
2000	11.17	0.40	0.20	0.30	0.35	0.35
4000	22.34	0.80	0.60	0.70	0.75	0.75
6000	33.50	1.30	0.90	1.10	1.15	1.15
8000	44.67	1.80	1.30	1.55	1.60	1.60
10000	55.84	2.30	1.80	2.05	2.10	2.10
12000	67.01	2.70	2.30	2.50	2.55	2.55
14000	78.18	3.20	2.70	2.95	3.00	3.00
16000	89.35	3.70	3.20	3.45	3.50	3.50
18000	100.51	4.30	3.70	4.00	4.05	4.05
20000	111.68	5.00	4.50	4.75	4.80	4.80
22000	122.85	5.50	5.50	5.50	5.55	5.55
24000	134.02	5.90	5.80	5.85	5.90	5.90
26000	145.19	6.60	6.20	6.40	6.45	6.45
28000	156.36	7.30	6.50	6.90	6.95	6.95
30000	167.52	8.00	7.20	7.60	7.65	7.65
32000	178.69	8.70	8.50	8.60	8.65	8.65
34000	189.86	9.90	9.10	9.50	9.55	9.55
36000	201.03	10.50	9.60	10.05	10.10	10.10
38000	212.20	11.50	10.30	10.90	10.95	10.95
40000	223.37	12.70	11.30	12.00	12.05	12.05
42000	234.53	14.00	12.30	13.15	13.20	13.20
44000	245.70	15.70	13.20	14.45	14.50	14.50
46000	256.87	17.70	15.10	16.40	16.45	16.45
48000	268.04	22.20	17.90	20.05	20.10	20.10
50000	279.21	25.00	23.00	24.00	24.05	24.05

ROTURA = Cmax/Area = 271.39 kg/cm^2

kg/cm^2

E1 = 40%ROTURA = 108.6 kg/cm^2
Eo = 15.9 kg/cm2
D1 = 4.5 x10^-4cm
Do = 0.5 x10^-4cm

$$M.E. = \frac{(E1 - Eo)}{(D1 - Do)x10^{-4}}$$

M.E. = 2.317 x10^5 kg/cm2

2.- RELACIÓN AGUA/CEMENTO 0.65

DOSIFICACIÓN DE LA FIBRA 35 Kg/m³ DE CONCRETO

PESO : 13.09 kg.
DIÁMETRO : 14.97 cm.
ÁREA : 176.0 cm2
CARGA MAX: 40600 kg.

Carga (kg)	Esfuerzo (kg/cm^2)	Lect. Izq.	Lect. Der.	Prom. Lec.	Lect. Correg.	Deform. Unit. ($x10^{-4}$) cm
0.00	0.00	0.20	0.00	0.10	0.00	0.00
2000	11.36	0.60	0.30	0.45	0.35	0.35
4000	22.73	1.00	0.70	0.85	0.75	0.75
6000	34.09	1.40	1.30	1.35	1.25	1.25
8000	45.45	1.80	1.70	1.75	1.65	1.65
10000	56.82	2.20	2.30	2.25	2.15	2.15
12000	68.18	2.50	2.60	2.55	2.45	2.45
14000	79.54	2.80	2.90	2.85	2.75	2.75
16000	90.90	3.50	3.40	3.45	3.35	3.35
18000	102.27	4.60	4.60	4.60	4.50	4.50
20000	113.63	5.20	5.30	5.25	5.15	5.15
22000	124.99	5.90	6.00	5.95	5.85	5.85
24000	136.36	6.60	6.90	6.75	6.65	6.65
26000	147.72	7.30	7.70	7.50	7.40	7.40
28000	159.08	8.10	8.70	8.40	8.30	8.30
30000	170.45	8.90	9.90	9.40	9.30	9.30
32000	179.37	9.90	10.20	10.05	9.95	9.95
34000	190.58	11.00	12.90	11.95	11.85	11.85
36000	201.79	12.60	15.10	13.85	13.75	13.75
38000	213.00	14.50	18.10	16.30	16.20	16.20
40000	224.22	18.00	22.40	20.20	20.10	20.10

ROTURA = Cmax/Area = 230.67 kg/cm^2

E1 = 40%ROTURA = 92.27 kg/cm^2
Eo = 14.77 kg/cm^2
D1 = 3.60 x10^-4cm
Do = 0.50 x10^-4cm

$$M.E. = \frac{(E1 - Eo)}{(D1 - Do)x10^{-4}}$$

M.E. = 2.500 x10^5 kg/cm^2

DOSIFICACIÓN DE LA FIBRA 45 Kg/m^3 DE CONCRETO

PESO : 13.7 kg.
DIÁMETRO : 14.98 cm.
ÁREA : 176.2 cm2
CARGA MAX: 43000 kg.

Carga (kg)	Esfuerzo (kg/cm^2)	Lect. Izq.	Lect. Der.	Prom. Lec.	Lect. Correg.	Deform. Unit. (x10^{-4}) cm
0	0.0	0.00	0.00	0.00	0.00	0.00
2000	11.35	0.50	0.30	0.40	0.40	0.40
4000	22.70	1.00	0.70	0.85	0.85	0.85
6000	34.04	1.40	1.10	1.25	1.25	1.25
8000	45.39	2.00	1.50	1.75	1.75	1.75
10000	56.74	2.50	1.90	2.20	2.20	2.20
12000	68.09	3.00	2.40	2.70	2.70	2.70
14000	79.44	3.60	2.90	3.25	3.25	3.25
16000	90.78	4.10	3.40	3.75	3.75	3.75
18000	102.13	4.70	3.90	4.30	4.30	4.30
20000	113.48	5.30	4.50	4.90	4.90	4.90
22000	124.83	6.00	5.00	5.50	5.50	5.50
24000	136.17	6.70	5.70	6.20	6.20	6.20
26000	147.52	7.50	6.40	6.95	6.95	6.95
28000	158.87	8.30	7.20	7.75	7.75	7.75
30000	170.22	9.20	8.00	8.60	8.60	8.60
32000	181.57	10.50	9.10	9.80	9.80	9.80
34000	192.91	11.70	10.20	10.95	10.95	10.95
36000	204.26	13.00	11.50	12.25	12.25	12.25
38000	215.61	14.70	13.20	13.95	13.95	13.95
40000	226.96	17.00	15.50	16.25	16.25	16.25
42000	238.31	21.00	18.90	19.95	19.95	19.95

ROTURA = Cmax/Area = 243.98 kg/cm^2

E1 = 40% ROTURA = 97.59 kg/cm^2
Eo = 14.00 kg/cm^2
D1 = 4.10 x10^-4cm
Do = 0.50 x10^-4cm

$$\text{M.E.} = \frac{(E1 - Eo)}{(D1 - Do)\text{x}10^{-4}}$$

M.E. = 2.322 x10^5 kg/cm2

DOSIFICACIÓN DE LA FIBRA 55 Kg/m^3 DE CONCRETO

PESO : 13.170 kg.
DIÁMETRO : 14.910 cm.
ÁREA : 174.601 cm2
CARGA MAX : 44800 kg.

Carga (kg)	Esfuerzo (kg/cm^2)	Lect. Izq.	Lect. Der.	Prom. Lec.	Lect. Correg.	Deform. Unit. (x10^{-4}) cm
0	0.0	0.00	0.00	0.00	0.00	0.00
2000	11.45	0.30	0.30	0.30	0.30	0.30
4000	22.91	0.60	0.60	0.60	0.60	0.60
6000	34.36	1.00	1.10	1.05	1.05	1.05
8000	45.82	1.50	1.70	1.60	1.60	1.60
10000	57.27	2.00	2.30	2.15	2.15	2.15
12000	68.73	2.40	2.90	2.65	2.65	2.65
14000	80.18	2.80	3.40	3.10	3.10	3.10
16000	91.64	3.80	4.00	3.90	3.90	3.90
18000	103.09	4.20	4.80	4.50	4.50	4.50
20000	114.55	5.30	5.30	5.30	5.30	5.30
22000	126.00	5.80	7.10	6.45	6.45	6.45
24000	137.46	6.20	7.90	7.05	7.05	7.05
26000	148.91	6.70	8.60	7.65	7.65	7.65
28000	160.37	7.50	9.60	8.55	8.55	8.55
30000	171.82	8.20	10.80	9.50	9.50	9.50
32000	183.28	9.00	12.20	10.60	10.60	10.60
34000	194.73	10.00	13.70	11.85	11.85	11.85
36000	206.18	11.00	15.60	13.30	13.30	13.30
38000	217.64	12.20	18.10	15.15	15.15	15.15
40000	229.09	14.00	22.90	18.45	18.45	18.45

ROTURA = Cmax/Area = 256.59 kg/cm^2

E1 = 40%ROTURA = 102.63 kg/cm^2
Eo = 19.10 kg/cm^2
D1 = 4.35 x10^-4cm
Do = 0.50 x10^-4cm

$$\text{M.E.} = \frac{(\text{E1} - \text{Eo})}{(\text{D1} - \text{Do})\text{x10\^{}-4}}$$

M.E. = 2.170 x10^5 kg/cm2

3.- RELACIÓN AGUA/CEMENTO 0.70

DOSIFICACIÓN DE LA FIBRA 35 Kg/m^3 DE CONCRETO

PESO : 13.7 kg.
DIAMETRO : 15 cm.
AREA : 176.7 cm2
CARGA MAX: 38000 kg.

Carga (kg)	Esfuerzo (kg/cm^2)	Lect. Izq.	Lect. Der.	Prom. Lec.	Lect. Correg.	Deform. Unit. (x10^{-4}) cm
0.00	0.00	0.00	0.00	0.00	0.00	0.00
2000	11.32	0.40	0.30	0.35	0.35	0.35
4000	22.64	0.90	0.90	0.90	0.90	0.90
6000	33.95	1.30	1.40	1.35	1.35	1.35
8000	45.27	1.80	1.90	1.85	1.85	1.85
10000	56.59	2.30	2.40	2.35	2.35	2.35
12000	67.91	2.70	3.00	2.85	2.85	2.85
14000	79.22	3.20	3.40	3.30	3.30	3.30
16000	90.54	3.70	3.90	3.80	3.80	3.80
18000	101.86	4.50	4.80	4.65	4.65	4.65
20000	113.18	5.00	5.50	5.25	5.25	5.25
22000	124.49	5.60	6.20	5.90	5.90	5.90
24000	135.81	6.50	7.10	6.80	6.80	6.80
26000	147.13	7.40	7.90	7.65	7.65	7.65
28000	158.45	8.40	8.90	8.65	8.65	8.65
30000	169.76	9.50	10.10	9.80	9.80	9.80
32000	179.37	11.20	11.50	11.35	11.35	11.35
34000	190.58	13.50	13.30	13.40	13.40	13.40
36000	201.79	16.50	15.20	15.85	15.85	15.85
38000	213.00	22.00	18.70	20.35	20.35	20.35

ROTURA = Cmax/Area = 215.04 kg/cm^2

E1 = 40%ROTURA = 86.01 kg/cm^2
Eo = 14.40 kg/cm^2
D1 = 3.50 x10^-4cm
Do = 0.50 x10^-4cm

$$\text{M.E.} = \frac{(\text{E1} - \text{Eo})}{(\text{D1} - \text{Do})\text{x}10^{-4}}$$

M.E. = 2.387 x10^5 kg/cm^2

DOSIFICACIÓN DE LA FIBRA 45 Kg/m^3 DE CONCRETO

PESO : 13.07 kg.
DIAMETRO : 14.96 cm.
AREA : 175.8 cm2
CARGA MAX: 41000 kg.

Carga (kg)	Esfuerzo (kg/cm^2)	Lect. Izq.	Lect. Der.	Prom. Lec.	Lect. Correg.	Deform. Unit. (x10^{-4}) cm
0.00	0.0	0.10	0.10	0.10	0.00	0.00
2000	11.38	0.30	0.30	0.30	0.20	0.20
4000	22.76	0.60	1.10	0.85	0.75	0.75
6000	34.13	1.30	1.80	1.55	1.45	1.45
8000	45.51	1.70	2.40	2.05	1.95	1.95
10000	56.89	2.50	3.20	2.85	2.75	2.75
12000	68.27	3.00	3.50	3.25	3.15	3.15
14000	79.65	3.40	3.80	3.60	3.50	3.50
16000	91.03	3.80	4.10	3.95	3.85	3.85
18000	102.40	5.00	5.80	5.40	5.30	5.30
20000	113.78	6.00	6.80	6.40	6.30	6.30
22000	125.16	7.00	7.80	7.40	7.30	7.30
24000	136.54	8.00	8.70	8.35	8.25	8.25
26000	147.92	9.00	9.80	9.40	9.30	9.30
28000	159.30	10.00	10.90	10.45	10.35	10.35
30000	170.67	12.00	12.30	12.15	12.05	12.05
32000	182.05	15.00	13.80	14.40	14.30	14.30
34000	193.43	18.00	15.60	16.80	16.70	16.70
36000	204.81	20.00	18.20	19.10	19.00	19.00
38000	216.19	22.00	20.10	21.05	20.95	20.95
40000	227.57	26.00	25.00	25.50	25.40	25.40

ROTURA = Cmax/Area = 233.25 kg/cm^2

E1 = 40% ROTURA = 93.30 kg/cm^2
Eo = 17.58 kg/cm^2
D1 = 4.00 x10^-4cm
Do = 0.50 x10^-4cm

$$M.E. = \frac{(E1 - Eo)}{(D1 - Do)x10^{-4}}$$

M.E. = 2.163 x10^5 kg/cm2

DOSIFICACIÓN DE LA FIBRA 55 Kg/m^3 DE CONCRETO

PESO : 13.0 kg.
DIAMETRO : 14.9 cm.
AREA : 174.6 cm2
CARGA MAX : 38200 kg.

Carga (kg)	Esfuerzo (kg/cm^2)	Lect. Izq.	Lect. Der.	Prom. Lec.	Lect. Correg.	Deform. Unit. (x10^{-4}) cm
0	0.0	0.00	0.10	0.05	0.00	0.00
2000	11.45	0.30	0.30	0.30	0.25	0.25
4000	22.91	0.60	0.80	0.70	0.65	0.65
6000	34.36	1.10	1.30	1.20	1.15	1.15
8000	45.82	1.60	1.90	1.75	1.70	1.70
10000	57.27	2.10	2.40	2.25	2.20	2.20
12000	68.73	2.70	3.00	2.85	2.80	2.80
14000	80.18	3.20	3.70	3.45	3.40	3.40
16000	91.64	3.60	4.20	3.90	3.85	3.85
18000	103.09	4.30	5.10	4.70	4.65	4.65
20000	114.55	4.80	5.90	5.35	5.30	5.30
22000	126.00	5.50	6.80	6.15	6.10	6.10
24000	137.46	6.40	7.70	7.05	7.00	7.00
26000	148.91	7.00	8.90	7.95	7.90	7.90
28000	160.37	7.70	10.10	8.90	8.85	8.85
30000	171.82	8.40	11.50	9.95	9.90	9.90
32000	183.28	9.50	13.70	11.60	11.55	11.55
34000	194.73	10.60	16.30	13.45	13.40	13.40
36000	206.18	12.00	20.40	16.20	16.15	16.15
38000	217.64	14.50	28.40	21.45	21.40	21.40

ROTURA = Cmax/Area = 218.78 kg/cm^2

E1 = 40%ROTURA = 87.5 kg/cm^2
Eo = 18.6 kg/cm^2
D1 = 3.8 x10^-4cm
Do = 0.5 x10^-4cm

$$\text{M.E.} = \frac{(\text{E1} - \text{Eo})}{(\text{D1} - \text{Do})\text{x10\^{}-4}}$$

M.E. = 2.088 x10^5 kg/cm^2

ANEXO D: ANÁLISIS DE COSTOS UNITARIOS

GENERALIDADES

En el presente anexo se presenta el análisis de costos para 1 m^3 de concreto, de los insumos empleados en la elaboración del concreto Patrón (sin fibra) y adicionando dosificaciones de Fibra de Acero Insonex de 35, 45, 55 kg/m^3 de concreto para las relaciones de agua/cemento: 0.60, 0.65, 0.70 respectivamente.

1. RELACIÓN AGUA/CEMENTO: 0.60

1.1 CONCRETO PATRÓN

Materiales	Und	Cantidad	P.U.	Parcial
Cemento Andino tipo I	bls	8.43	17.50	147.53
Arena Gruesa	m^3	0.345	20.00	6.90
Piedra Chancada de 3/4"	m^3	0.315	40.00	12.60
Agua	lts	225.5	0.005	1.13
Costo Total (S/.)				**168.15**

1.2 DOSIFICACIÓN DE FIBRA: 35 kg/m^3 DE CONCRETO

Materiales	Und	Cantidad	P.U.	Parcial
Cemento Andino tipo I	bls	8.43	17.50	147.53
Arena Gruesa	m^3	0.345	20.00	6.90
Piedra Chancada de 3/4"	m^3	0.315	40.00	12.60
Agua	lts	225.5	0.005	1.13
Fibra de Acero Insonex	kg	35.00	5.60	196.00
Costo Total (S/.)				**364.15**
% Respecto al Patrón		**216.56**	%	

1.3 DOSIFICACIÓN DE FIBRA: 45 kg/m^3 DE CONCRETO

Materiales	Und	Cantidad	P.U.	Parcial
Cemento Andino tipo I	bls	8.43	17.50	147.53
Arena Gruesa	m^3	0.345	20.00	6.90
Piedra Chancada de 3/4"	m^3	0.315	40.00	12.60
Agua	lts	225.5	0.005	1.13
Fibra de Acero Insonex	kg	45.00	5.60	252.00
Costo Total (S/.)				**420.15**
% Respecto al Patrón		**249.86**	%	

1.4 DOSIFICACIÓN DE FIBRA: 55 kg/m^3 DE CONCRETO

Materiales	Und	Cantidad	P.U.	Parcial
Cemento Andino tipo I	bls	8.43	17.50	147.53
Arena Gruesa	m^3	0.345	20.00	6.90
Piedra Chancada de 3/4"	m^3	0.315	40.00	12.60
Agua	lts	225.5	0.005	1.13
Fibra de Acero Insonex	kg	55.00	5.60	308.00
Costo Total (S/.)				**476.15**
% Respecto al Patrón		**283.17**	%	

2. RELACIÓN AGUA/CEMENTO: 0.65

2.1 CONCRETO PATRÓN

Materiales	Und	Cantidad	P.U.	Parcial
Cemento Andino tipo I	bls	7.75	17.50	135.63
Arena Gruesa	m^3	0.351	20.00	7.02
Piedra Chancada de 3/4"	m^3	0.320	40.00	12.80
Agua	lts	224.6	0.005	1.12
Costo Total (S/.)				**156.57**

2.2 DOSIFICACIÓN DE FIBRA: 35 kg/m^3 DE CONCRETO

Materiales	Und	Cantidad	P.U.	Parcial
Cemento Andino tipo I	bls	7.75	17.50	135.63
Arena Gruesa	m^3	0.351	20.00	7.02
Piedra Chancada de 3/4"	m^3	0.320	40.00	12.80
Agua	lts	224.6	0.005	1.12
Fibra de Acero Insonex	kg	35.00	5.60	196.00
Costo Total (S/.)				**352.57**
% Respecto al Patrón		**225.19**	%	

2.3 DOSIFICACIÓN DE FIBRA: 45 kg/m^3 DE CONCRETO

Materiales	Und	Cantidad	P.U.	Parcial
Cemento Andino tipo I	bls	7.75	17.50	135.63
Arena Gruesa	m^3	0.351	20.00	7.02
Piedra Chancada de 3/4"	m^3	0.320	40.00	12.80
Agua	lts	224.6	0.005	1.12
Fibra de Acero Insonex	kg	45.00	5.60	252.00
Costo Total (S/.)				**408.57**
% Respecto al Patrón		**260.95**	%	

2.4 DOSIFICACIÓN DE FIBRA: 55 kg/m^3 DE CONCRETO

Materiales	Und	Cantidad	P.U.	Parcial
Cemento Andino tipo I	bls	7.75	17.50	135.63
Arena Gruesa	m^3	0.351	20.00	7.02
Piedra Chancada de 3/4"	m^3	0.320	40.00	12.80
Agua	lts	224.6	0.005	1.12
Fibra de Acero Insonex	kg	55.00	5.60	308.00
Costo Total (S/.)				**464.57**
% Respecto al Patrón		**296.72**	%	

3. RELACIÓN AGUA/CEMENTO: 0.70

3.1 CONCRETO PATRÓN

Materiales	Und	Cantidad	P.U.	Parcial
Cemento Andino tipo I	bls	7.16	17.50	125.30
Arena Gruesa	m^3	0.356	20.00	7.12
Piedra Chancada de 3/4"	m^3	0.324	40.00	12.96
Agua	lts	223.8	0.005	1.12
Costo Total (S/.)				**146.50**

3.2 DOSIFICACIÓN DE FIBRA: 35 kg/m^3 DE CONCRETO

Materiales	Und	Cantidad	P.U.	Parcial
Cemento Andino tipo I	bls	7.16	17.50	125.30
Arena Gruesa	m^3	0.356	20.00	7.12
Piedra Chancada de 3/4"	m^3	0.324	40.00	12.96
Agua	lts	223.8	0.005	1.12
Fibra de Acero Insonex	kg	35.00	5.60	196.00
Costo Total (S/.)				**342.50**
% Respecto al Patrón		**233.79**	%	

3.3 DOSIFICACIÓN DE FIBRA: 45 kg/m^3 DE CONCRETO

Materiales	Und	Cantidad	P.U.	Parcial
Cemento Andino tipo I	bls	7.16	17.50	125.30
Arena Gruesa	m^3	0.356	20.00	7.12
Piedra Chancada de 3/4"	m^3	0.324	40.00	12.96
Agua	lts	223.8	0.005	1.12
Fibra de Acero Insonex	kg	45.00	5.60	252.00
Costo Total (S/.)				**398.50**
% Respecto al Patrón		**272.01**	%	

3.4 DOSIFICACIÓN DE FIBRA: 55 kg/m^3 DE CONCRETO

Materiales	Und	Cantidad	P.U.	Parcial
Cemento Andino tipo I	bls	7.16	17.50	125.30
Arena Gruesa	m^3	0.356	20.00	7.12
Piedra Chancada de 3/4"	m^3	0.324	40.00	12.96
Agua	lts	223.8	0.005	1.12
Fibra de Acero Insonex	kg	55.00	5.60	308.00
Costo Total (S/.)				**454.50**
% Respecto al Patrón		**310.24**	%	

ANEXO E: EXPOSICIÓN DE FOTOS

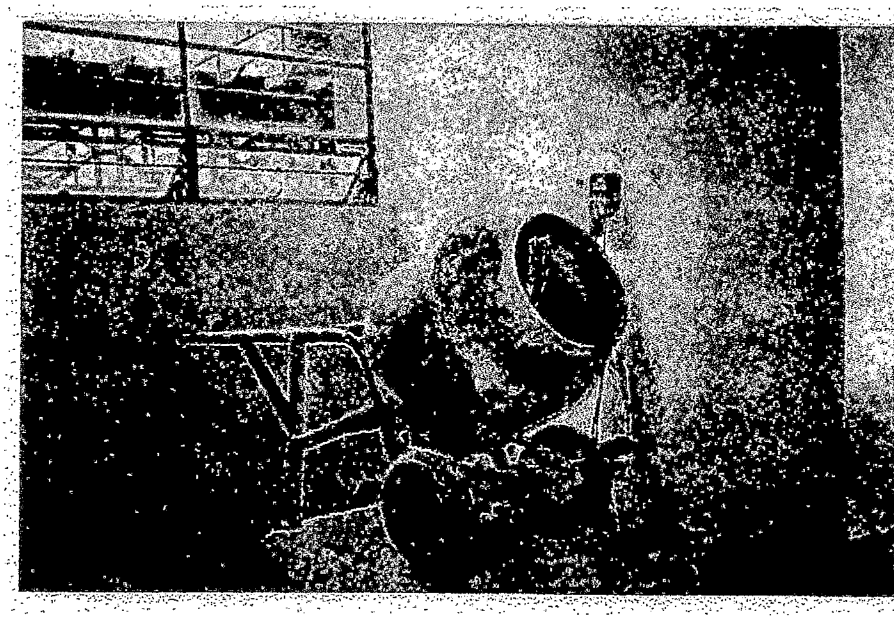


FOTO N° 01 MEZCLADORA UTILIZADA

FOTO N° 02 ENSAYO DE FLUIDEZ

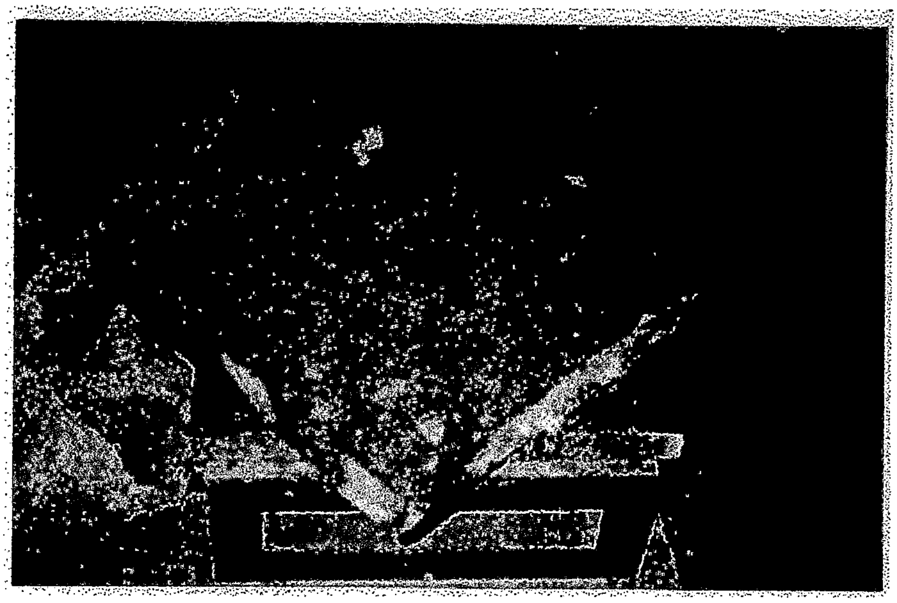


FOTO N° 03 VIBRADO DEL CONCRETO

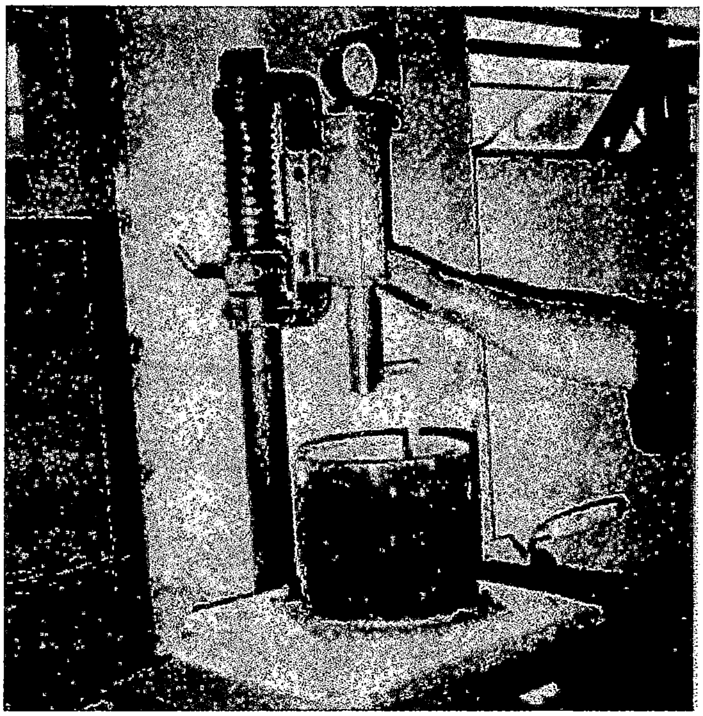


FOTO N° 04 ENSAYO DE TIEMPO DE FRAGUADO

FOTO N° 05 ENSAYO DE CONTENIDO DE AIRE

FOTO N° 06 ENSAYO DE PESO UNITARIO COMPACTADO

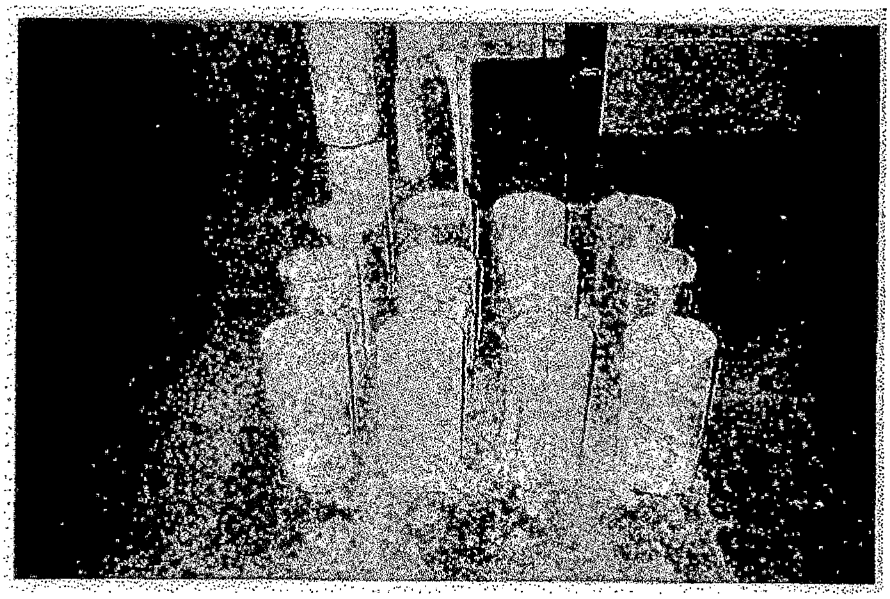


FOTO N° 07 PROBETAS ENSAYADAS

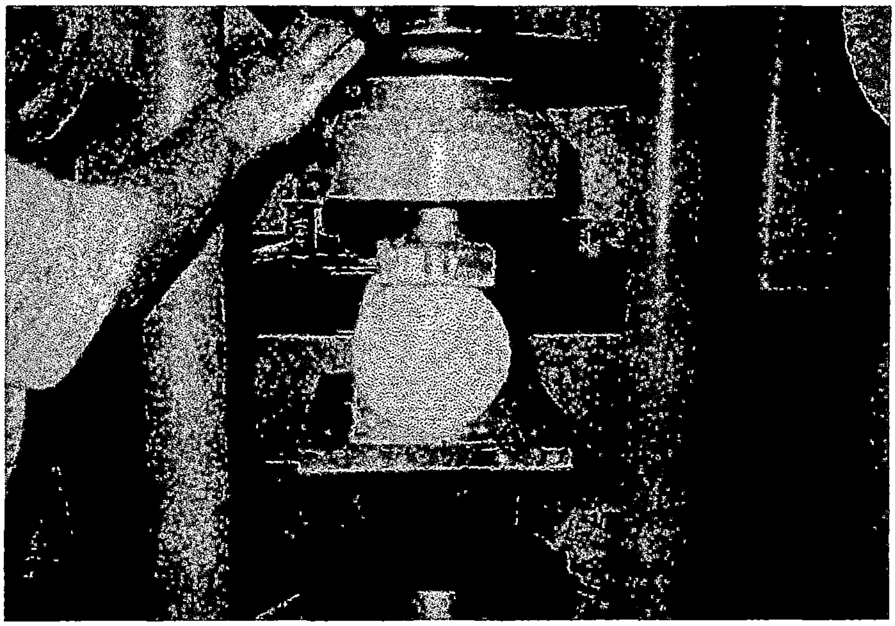


FOTO N° 08 ENSAYO DE COMPRESIÓN DIAMETRAL

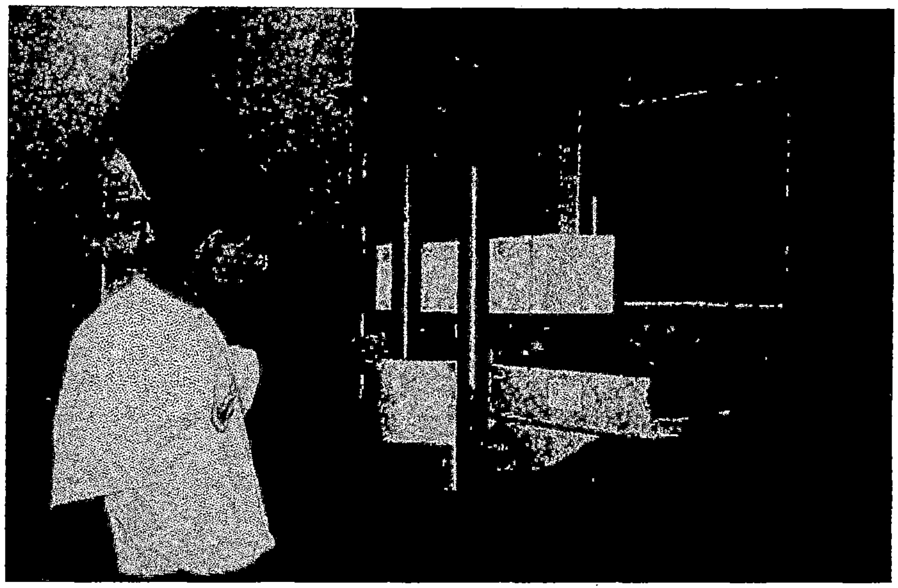


FOTO N° 09 ENSAYO DE FLEXIÓN EN VIGAS

FOTO N° 10 VIGA CON FIBRA DE ACERO

FOTO N° 11 VIGA SIN FIBRA DE ACERO

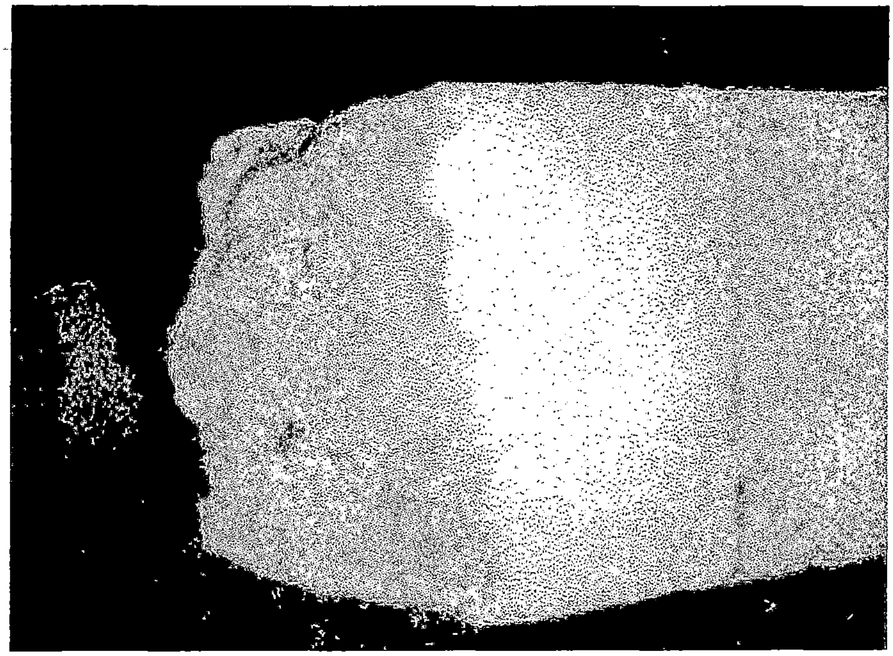


FOTO N° 12 CORTE DE LA VIGA

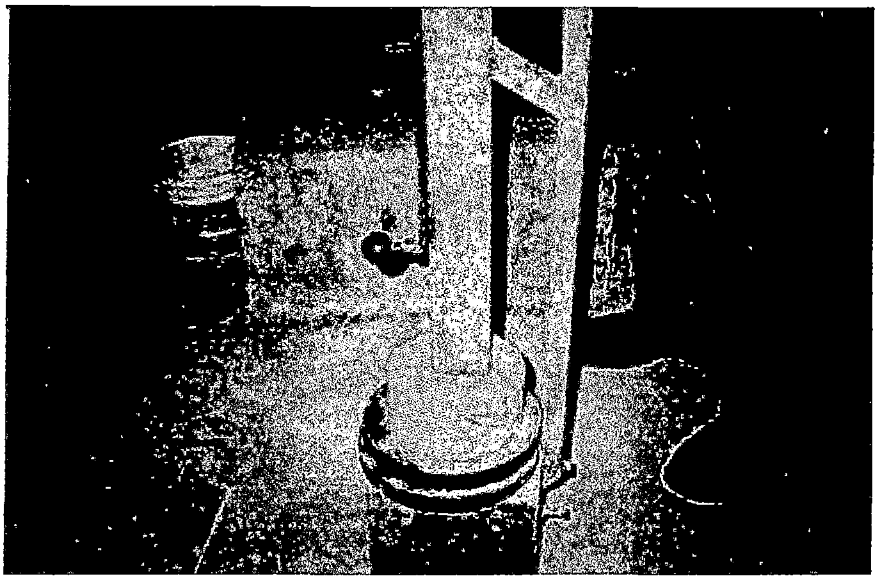


FOTO N° 13 ENSAYO DE IMPACTO EN DISCOS

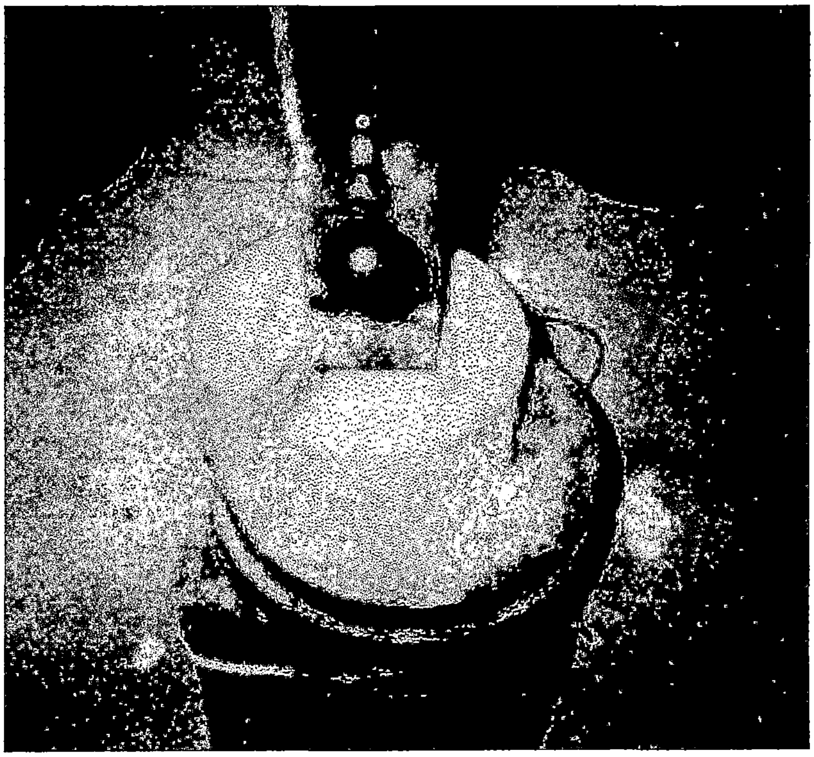


FOTO N° 14 FALLA DEL DISCO

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AUTOR : ANGEL AVENDAÑO ARONI
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BIBLIOTECA	:	FIC - UNI
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CONTENIDO	:	ANÁLISIS DE LOS RESULTADOS, CONCLUSIONES RECOMENDACIONES

TÍTULO	:	ESTUDIO DE LASP ROPIEDADES DEL CONCRETO UTILIZANDO FIBRAS DE REFUERZO DE ACERO
AUTOR	:	JULIO CUARESMA CARBAJAL
BIBLIOTECA	:	FIC - UNI
LUGAR – AÑO	:	LIMA - PERU 2001
CONTENIDO	:	ANÁLISIS DE LOS RESULTADOS, CONCLUSIONES RECOMENDACIONES

NORMAS

CONCRETO

NTP 339.035:1999	Método de ensayo para determinar el Asentamiento del concreto con el cono de Abrams.
NTP 339.077:1981	Método de ensayo para determinar la Exudación del concreto.
NTP 339.035:1979	Método de ensayo para determinar el Peso Unitario Compactado.
NTP 339.082:2001	Método de ensayo para determinar el Tiempo de Fraguado en el concreto por medio de su resistencias a la penetración.
NTP 339.085:1981	Método de ensayo para determinar el Índice de Consistencia mediante el método de sacudidas.
NTP 339.080:1981	Método por presión para determinar el Contenido de Aire en mezclas frescas.
NTP 339.034:1999	Método de ensayo para determinar el esfuerzo a compresión de probetas cilíndricas.
NTP 339.084:1981	Método de ensayo para determinar la resistencia a la tracción por Compresión Diametral de una probeta cilíndrica
ASTM C496: 1963	Método de ensayo para determinar el Módulo de Elasticidad Estático.
ACI - 542	Método de ensayo para determinar la resistencia al Impacto.

AGREGADOS

NTP 400.012	Análisis Granulométrico, Modulo de Finura, Superficie Específica.
NTP 400.017	Método de ensayo para determinar el Peso Unitario Suelto y Compactado.
NTP 400.021	Método de ensayo para determinar el Peso Específico y la Absorción del Agregado Fino.
NTP 400.022	Método de ensayo para determinar el Peso Específico y la Absorción del Agregado Grueso.
ASTM C - 566	Método de ensayo para determinar el Contenido de Humedad.

Designation: A 820 – 90

AMERICAN SOCIETY FOR TESTING AND MATERIALS
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Standard Specification for Steel Fibers for Fiber Reinforced Concrete[1]

This standard is issued under the fixed designation A 820; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This specification covers minimum standards for steel fibers intended for use in fiber reinforced concrete. Steel fibers for this purpose are defined as pieces of smooth or deformed cold drawn wire; smooth or deformed cut sheet; melt-extracted fibers; or other steel fibers that are sufficiently small to be dispersed at random in a concrete mixture.

1.2 This specification provides for measurement of dimensions, tolerances from specified dimensions, and required minimum physical properties, and prescribes testing cedures to establish conformance to these requirements.

1.3 The values stated in inch-pound units are to be regarded as the standard.

2. Referenced Documents

2.1 The following documents of the issue in effect on the date of material purchase form a part of this specification to the extent referenced herein.

2.2 *ASTM Standards:*

A 370 Methods and Definitions for Mechanical Testing of Steel Products[2]

C 94 Specification for Ready-Mixed Concrete[3]

2.3 *ACI Standard:*

544.1 R-82 State-of-the-Art Report on Fiber Reinforced Concrete[4]

3. Terminology

3.1 *Symbols*—The following symbols used in this specification are defined as:

3.1.1 A—cross-sectional area, in.^{2} (mm^2).

3.1.2 d—diameter, in. (mm).

3.1.3 f_u—ultimate tensile strength, psi (MPa).

3.1.4 l—length, in. (mm).

3.1.5 λ—l/d = aspect ratio.

3.1.6 The subscript "n" on dimensional units indicates "nominal" and the subscript "e" indicates "equivalent". "Nominal" and "equivalent" dimensions are calculated from other measurable dimensions or average weights.

4. Classification

4.1 Four general types of steel fibers are identified in this specification based upon the product used as a source of the steel fiber material.

4.1.1 Type I, cold drawn wire.

4.1.2 Type II, cut sheet.

4.1.3 Type III, melt-extracted.

4.1.4 Type IV, other fibers.

4.2 Fibers may be straight or deformed.

5. Ordering Information

5.1 Orders for material under this specification should include the following:

5.1.1 ASTM designation and year of issue,

5.1.2 Quantity in pounds or tons,

5.1.3 Type or types permissible (4.1),

5.1.4 Diameter or equivalent diameter (8.1), or range of equivalent diameters (8.1.5),

5.1.5 Length or nominal length (8.1),

5.1.6 Deformations, if required, and

5.1.7 Whether certification by the producer or supplier is required including whether a report is to be furnished (15.1).

NOTE 1—For information on satisfactory sizes and aspect ratios, see ACI 544.1R-82, and contact producer regarding availability.

6. Material and Manufacture

6.1 The materials and manufacturing methods used shall be such that the fibers produced conform to the requirements in this specification.

7. Responsibility for Quality Assurance

7.1 *Responsibility for Inspection*—Unless otherwise specified in the contract or purchase order, the producer is responsible for the performance of all inspection and test requirements specified herein. Except as otherwise specified in the contract or order, the producer may use his own or any other suitable facility for the performance of the inspection and test requirements specified herein unless disapproved by the purchaser. The purchaser shall have the right to perform any of the inspections and tests set forth in this specification where such inspections are deemed necessary to assure that material conforms to prescribed requirements.

8. Dimensions and Tolerances

8.1 *Dimensions:*

8.1.1 Straight cold-drawn wire fibers are specified by diameter (d) or equivalent (d_e) and length (l), that establish a specified aspect ratio (l/d) or (l/d_e).

8.1.2 Deformed cold-drawn wire fibers are specified by the diameter (d) or equivalent diameter (d_e) and length (out-to-out) after bending (l_n). Nominal aspect ratio (λ_n) is established as (l_n/d) or (l_n/d_e). See Fig. 1.

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[1] This specification is under the jurisdiction of ASTM Committee A-1 on Steel, Stainless Steel, and Related Alloys and is the direct responsibility of Subcommittee A01.05 on Steel Reinforcement.

Current edition approved Apr. 27, 1990. Published June 1990. Originally published as A 820 – 85. Last previous edition A 820 – 85.

[2] *Annual Book of ASTM Standards*, Vol 01.04.

[3] *Annual Book of ASTM Standards*, Vol 04.02.

[4] Available from American Concrete Institute, P.O. Box 19150, Redford Station, Detroit, MI 48219.

8.1.3 Cut sheet fibers are specified by thickness (t), width (w), and length (l). Nominal aspect ratio (λ_n) can be computed as $l/\sqrt{4A/\pi} = l/d_e$, where $A = tw$ and d_e = equivalent diameter. See Fig. 2.

8.1.4 Deformed cut sheet fibers are specified by thickness (t), width (w), and out-to-out length after deformation (l_n). Nominal aspect ratio (λ_n) can be computed as $l_n\sqrt{4A/\pi} = l_n/d_e$. See Fig. 3.

8.1.5 Melt-extracted or other fibers are specified by a range of equivalent diameters, (d_e), and length (l). Equivalent diameter is computed from measured average length and the weight of a known quantity of fibers, based upon 0.2836 lb/in.^{3} (7850 kg/m^3). See Fig. 4.

8.2 *Tolerances:*

8.2.1 The length shall not vary from its specified value more than ±10 %.

8.2.2 The diameter or equivalent diameter shall not vary from its specified value more than ±10 %.

8.2.3 The aspect ratio shall not vary from its specified value more than ±15 %.

9. Tensile Requirements

9.1 The average tensile strength, f_u, shall not be less than 50 000 psi (345 MPa).

10. Bending Requirements

10.1 Fibers shall withstand bending around a 0.125-in. (3.18-mm) inside diameter to an angle of 90° at temperatures not less than 60°F (16°C) without breaking.

NOTE 2—The bending requirements of this specification provide a general indication of fiber ductility, as may be important in resisting breakage during handling and mixing operations. Ductility measures of fiber reinforced concrete are outside the scope of this specification; see ACI 544.1R-82.

11. Surface Condition

11.1 Seams and surface irregularities shall not be cause for rejection provided that tensile properties are not less than requirements of this specification and mixing performance in concrete is not adversely affected.

11.2 Rust, mill scale, or other coatings shall not be cause for rejection provided that the individual fibers separate when mixed in concrete in accordance with Specification C 94, and tensile and bending properties are not less than the requirements of this specification.

12. Measurement of Dimensions

12.1 Measurement of dimensions shall be performed on not less than 10 randomly selected specimens for each test to establish the average for conformance to specified tolerances. At least 90 % of the specimens in each test shall meet the specified tolerances for length, diameter or equivalent diameter, and aspect ratio.

12.2 At least one test shall be performed for each 5 tons

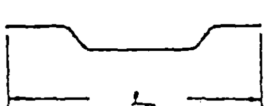

FIG. 1 Deformed Cold-Drawn Fibers

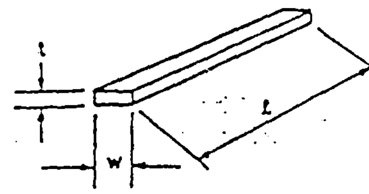

FIG. 2 Cut Sheet Fibers

(4.5 Mg) of material or each shipment if less than 5 tons (4 Mg).

13. Tests

13.1 At least one tensile test, consisting of 10 randon selected finished fibers, shall be performed for each 5 to (4.5 Mg) of material or each shipment if less than 5 tons (4 Mg). The average value of f_u in these tests must not be le than 50 000 psi (345 MPa). The tensile strength of any one the ten specimens shall not be less than 45 000 psi (3 MPa). Where the parent source material consists of sheet wire, tensile tests by the producer may be performed larger samples of source material. One sample of ea different source material used shall then be tested for each tons (4.5 Mg) of material or each shipment if less than 5 tc (4.5 Mg). The tensile strength of a single sample of sour material shall not be less than 50 000 psi (345 MPa).

13.1.1 The cross-sectional area used to compute t tensile strength shall be carried out to five decimal places, units of square inches, and shall be: *(1) for drawn wire fibe* the area calculated from the actual diameter of the pare source material or finished fibers; *(2) for cut sheet fibers,* t area calculated from the actual thickness and width of t parent source specimen, or if fibers are tested, the area each individual fiber calculated from the measured leng and weight of the fiber, weighed to the nearest 0.0001 based on a density of 0.2836 lb/in.^{3} (7850 kg/m^3); and *(3) melt-extraction fibers or other fibers specified by equival diameter,* the area calculated from the equivalent diameter the fibers. See 8.1.5. The breaking load in pounds-force individual fibers shall be measured to at least three sign cant figures. Testing shall be in accordance with Methc and Definitions A 370, where applicable.

13.2 Ten randomly selected specimens of finished fib shall be bent 90° around a 0.125-in. (3.18-mm) ins diameter without breaking. The test may be done by ha At least one test consisting of 10 specimens shall be made each 5 tons (4.5 Mg) of material or each shipment if less th 5 tons (4.5 Mg). At least 90 % of the specimens must pass test.

14. Rejection and Retest

14.1 If any test fails to conform to the requirements of t specification, it shall be cause for rejection of the mater represented by the test. When any test fails to meet t requirements of tension, bending, or dimensional toleranc a retest will be allowed. This retest shall be performed twice the number of randomly selected specimens origina tested. The retest shall meet the requirements of the spec cation or the lot shall be rejected.

14.2 Also, material in which defects are discovered dur

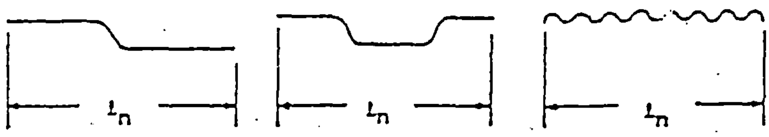

FIG. 3 Deformed Cut Sheet Fibers

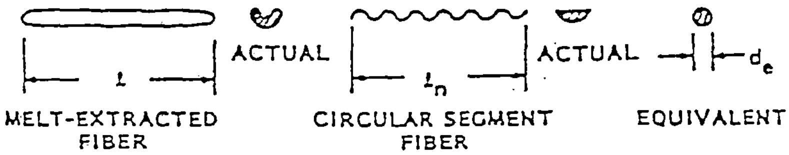

FIG. 4 Melt-Extracted and Other Fibers

subsequent manufacturing operation may be rejected. If rejected, the producer or supplier shall be responsible only for replacement of material to the purchaser. As much as possible of the rejected material shall be returned to the producer or supplier.

14.3 Rejection shall be reported to the producer or supplier promptly and in writing. In case of dissatisfaction with the results of the test, the producer or supplier may make claim for a rehearing.

15. Certification

15.1 The producer or supplier shall on request furnish to the purchaser a certificate stating that each lot has been sampled, tested, and inspected in accordance with this specification and has met the requirements. When specified in the purchase order or contract, a report of the test results shall be furnished.

16. Packaging and Marking

16.1 The material shall be packaged to provide adequate protection during normal handling and transportation and each package shall contain only one type and size of material unless otherwise agreed upon. The type of packaging and gross weight of containers shall, unless otherwise agreed upon, be at the producer's or supplier's discretion provided that they are such as to ensure acceptance by common or other carriers for safe transportation at the lowest rate to the delivery point.

16.2 Each shipping container shall be marked with the purchase order number, material, size, type, specification designation, net weight, and the producer's name or trademark.

ORIGINAL

Designation: C 1116 – 89

AMERICAN SOCIETY FOR TESTING AND MATERIALS
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Standard Specification for Fiber-Reinforced Concrete and Shotcrete[1]

This standard is issued under the fixed designation C 1116; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This specification covers all forms of fiber-reinforced concrete that are delivered to a purchaser with the ingredients uniformly mixed, and that can be sampled and tested at the point of delivery. It does not cover the placement, consolidation, curing, or protection of the fiber-reinforced concrete after delivery to the purchaser.

1.2 Certain sections of this specification are also applicable to fiber-reinforced concrete intended for shotcreting by the dry-mix process when sampling and testing of concrete is possible only at the point of placement. In this case, the sections dealing with batching plant, mixing equipment, mixing and delivery, and measurement of workability and air content, are not applicable.

1.3 This specification does not cover thin-section glass fiber-reinforced concrete manufactured by the spray-up process that is under the jurisdiction of ASTM Subcommittee C27.40.

1.4 The values stated in inch-pound units are to be regarded as the standard.

1.5 The following precautionary statement pertains only to the test method portion, Sections 16 and 19, of this specification: *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

A 820 Specification for Steel Fibers for Fiber-Reinforced Concrete[2]
C 31 Practice of Making and Curing Concrete Test Specimens in the Field[3]
C 33 Specification for Concrete Aggregates[3]
C 39 Test Method for Strength of Cylindrical Concrete Specimens[3]
C 42 Methods of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete[3]
C 78 Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)[3]
C 94 Specification for Ready-Mixed Concrete[3]
C 109 Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-/in. or 50-/mm Cube Specimens)[4]
C 138 Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete[3]
C 143 Test Method for Slump of Portland Cement Concrete[3]
C 150 Specification for Portland Cement[2]
C 172 Method of Sampling Freshly Mixed Concrete[3]
C 173 Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method[3]
C 191 Test Method for Time of Setting of Hydraulic Cement by Vicat Needle[4]
C 192 Practice of Making and Curing Concrete Test Specimens in the Laboratory[3]
C 231 Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method[3]
C 260 Specification for Air-Entraining Admixtures for Concrete[3]
C 330 Specification for Lightweight Aggregates for Structural Concrete[3]
C 387 Specification for Packaged, Dry, Combined Materials for Mortar and Concrete[3]
C 494 Specification for Chemical Admixtures for Concrete[3]
C 567 Test Method for Unit Weight of Structural Lightweight Concrete[3]
C 595 Specification for Blended Hydraulic Cements[3]
C 618 Specification for Fly Ash and Raw or Calcined Natural Pozzolans for Use as a Mineral Admixture in Portland Cement Concrete[3]
C 637 Specification for Aggregates for Radiation-Shielding Concrete[3]
C 666 Test Method for Resistance of Concrete to Rapid Freezing and Thawing[3]
C 684 Method of Making, Accelerated Curing, and Testing of Concrete Compression Test Specimens[3]
C 685 Specification for Concrete Made by Volumetric Batching and Continuous Mixing[3]
C 887 Specification for Packaged, Dry, Combined Materials for Surface Bonding Mortar[5]
C 995 Test Method for Time of Flow of Fiber-Reinforced Concrete Through Inverted Slump Cone[3]
C 1017 Specification for Chemical Admixtures for Use in Producing Flowing Concrete[3]
C 1018 Test Method for Flexural Toughness and First-Crack Strength of Fiber-Reinforced Concrete (Using

[1] This specification is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03.04 on Fiber Reinforced Concrete.

Current edition approved June 15, 1989. Published July 1989.

[2] *Annual Book of ASTM Standards*, Vol 01.04.

[3] *Annual Book of ASTM Standards*, Vol 04.02.

[4] *Annual Book of ASTM Standards*, Vol 04.01.

[5] *Annual Book of ASTM Standards*, Vol 04.05.

Beam with Third-Point Loading)[3]

C 1077 Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Laboratory Evaluation[3]

D 512 Test Methods for Chloride Ion in Water[5]

D 516 Test Methods for Sulfate Ion in Water[6]

2.2 *ACI Standards and Reports:*

211.1 Standard Practice for Selecting Proportions for Normal and Heavyweight Concrete[7]

211.2 Standard Practice for Selecting Proportions for Structural Lightweight Concrete[7]

214 Recommended Practice for Evaluation of Strength Test Results of Concrete[7]

506.1R, State-of-the-Art Report on Fiber-Reinforced Shotcrete[7]

506.2 Specification for Materials, Proportioning and Application of Shotcrete[7]

506 R, Guide for Shotcreting[7]

544.3R Guide for Specifying, Mixing, Placing and Finishing Steel Fiber-Reinforced Concrete[7]

2.3 *AASHTO Standard:*

T26 Test Method for Solids Content of Wash Water[8]

3. Terminology

3.1 *Descriptions of Terms Specific to This Standard:*

3.1.1 *fibers*—slender and elongated filaments in the form of bundles, networks, or strands of any natural or manufactured material that can be distributed throughout freshly mixed concrete.

3.1.2 *manufacturer*—the contractor, subcontractor, supplier, or producer who furnishes the fiber-reinforced concrete.

3.1.3 *purchaser*—the owner or representative thereof.

4. Classification

4.1 This specification classifies fiber-reinforced concrete or shotcrete by the material type of the fiber incorporated. The performance of a fiber-reinforced concrete or shotcrete depends strongly upon the susceptibility of the fibers to physical damage during the mixing or shotcreting process, their chemical compatibility with the normally alkaline environment within cement paste, and their resistance to service conditions encountered within uncracked concrete or as a consequence of cracking, involving, for example, carbon dioxide, chlorides or sulphates in solution with water and oxygen or ultraviolet light in the atmosphere. The magnitude of improvements in the mechanical properties of the concrete or shotcrete imparted by fibers also reflects the material characteristics of the fiber type with fibers having a high modulus of elasticity and tensile strength being more effective on an equivalent volume basis than fibers of low modulus and strength.

4.1.1 *Type I Steel Fiber-Reinforced Concrete or Shotcrete*—Contains stainless steel, alloy steel, or carbon steel fibers, (see Note 1).

NOTE 1—Steel fibers are not easily damaged by the mixing or shotcreting processes and are chemically compatible with the normally alkaline environment within cement paste. Carbon steel fibers will rust under conditions that cause rusting of conventional steel, for example, in the near-surface portion of concrete subject to carbonation.

4.1.2 *Type II Glass Fiber-Reinforced Concrete or Shotcrete*—Contains alkali-resistant glass fibers, (see Note 2).

NOTE 2—Glass fibers in concrete or shotcrete subjected to wetting, humid atmosphere, or contact with moist ground have the potential to react with the alkalies present in cement paste thereby weakening the fibers. They also tend to become embrittled by hydration products penetrating the fiber bundles and filling the interstitial spaces between the individual glass filaments. Both mechanisms cause reductions in strength, toughness, and impact resistance with age. The alkali-resistant (AR) types of glass fiber developed for use with cement are more resistant to alkalies than the E-glass and other types not marketed specifically for use in cement, and should be used in conjunction with established techniques for suppressing the alkali-silica reaction, for example, use of a low-alkali cement or a mineral admixture, or both. However, even the use of AR-glass fibers does not prevent deterioration in glass fiber-reinforced concrete exposed to moisture for a long period of time, but only slows the rate at which it occurs.

Glass fibers can be damaged by conventional concrete mixing processes employing coarse aggregate, but have been used in shotcrete and in other cementitious matrices such as mechanically mixed masonry mortar (see Specification C 887) and thin-section glass fiber-reinforced concrete prepared by the spray-up process (under the jurisdiction of ASTM Subcommittee C27.40).

4.1.3 *Type III Synthetic Fiber-Reinforced Concrete or Shotcrete*—Contains virgin homopolymer polypropylene fibers or other synthetic fibers for which documentary evidence can be produced confirming their long-term resistance to deterioration when in contact with the moisture and alkalies present in cement paste or the substances present in air-entraining and chemical admixtures, (see Note 3 and 4.2).

NOTE 3—Virgin homopolymer polypropylene fibers are not attacked by the constituents of cement or the substances encountered in most air-entraining and chemical admixtures. Fibers made with some other polymers may deteriorate when in contact with moisture, alkalies, the detergents present in some air-entraining admixtures, or some of the ingredients of chemical admixtures.

4.2 When the purchaser chooses to permit the use of fibers other than those complying with the classifications in 4.1, for example: natural fibers, metallic fibers other than steel, carbon fibers, etc., the producer shall show evidence satisfactory to the purchaser that the type of fiber proposed for use does not react adversely with the concrete or shotcrete matrix, including the constituents of any admixtures present, or with the surrounding environment in the cracked matrix, causing deterioration in mechanical properties with age under the exposure conditions anticipated in the application.

5. Basis of Purchase

5.1 The basis of purchase for conventionally mixed fiber-reinforced concrete shall be the cubic yard or cubic metre o freshly mixed and unhardened material as discharged from the mixer.

5.2 The volume of freshly mixed and unhardened material in a given batch shall be determined from the total weight of the batch divided by the unit weight in pounds pe cubic foot or kilograms per cubic metre. The total weight o the batch shall be calculated either as the sum of the weight

[6] *Annual Book of ASTM Standards*, Vol 11.01.

[7] Available from American Concrete Institute, PO Box 19150, Detroit, MI, 48219.

[8] Available from American Association of State Highway and Transportation Officials, Washington DC.

of all materials, including water, entering the batch, or as the net weight of the concrete in the batch as delivered. The unit weight shall be determined in accordance with Method C 138 or C 567 for the average of at least three measurements, each on a different sample. Sampling shall be in accordance with Practice C 172.

NOTE 4—It should be understood that the volume of hardened concrete may be, or may appear to be, less than expected due to waste and spillage, over-excavation, spreading forms, some loss of entrained air, or settlement of wet mixtures, none of which are the responsibility of the manufacturer.

5.3 The basis of purchase for fiber-reinforced shotcrete shall normally be the cubic yard or cubic metre. For wet-mix shotcrete, the volume shall be calculated from the quantities delivered and the unit weight. For dry-mix shotcrete, the volume shall be calculated from the weights of constituent materials mixed and their respective specific gravities. At the option of the purchaser, where the surface to be shotcreted is plane and a uniform finished thickness of shotcrete is specified, the basis of purchase shall be the square yard or square metre.

6. Ordering Information

6.1 The purchaser shall specify the following:

6.1.1 Type of cement at the purchaser's option, otherwise the cement shall be Type I meeting the requirements of Specification C 150;

6.1.2 Types of fine and coarse aggregate at the purchaser's option, otherwise the aggregates shall be normal weight meeting the requirements of Specification C 33;

6.1.3 Slump or time of flow required at the point of delivery, or when appropriate the point of placement, subject to the tolerances hereinafter specified;

6.1.3.1 Slump shall be specified when it is anticipated to be 2 in. (50 mm) or more, and time of flow shall be specified when slump is anticipated to be less than 2 in. (50 mm). Slump or time of flow shall not be specified for shotcrete placed by the dry process.

NOTE 5—The time of flow of fiber-reinforced concrete through an inverted slump cone, determined in accordance with Method C 995, is a better indicator than slump (Method C 143) of the appropriate level of workability for fiber-reinforced concrete placed by vibration because such concrete can exhibit very low slump due to the presence of fibers and still be easily consolidated. Mixtures with a time of flow of 8 to 15 s are readily consolidated by vibration. Consolidation becomes more difficult with increase in time of flow, and is extremely difficult even when using internal vibration if the time of flow exceeds 30 s. Mixtures with a time of flow less than 8 s should be evaluated in terms of slump because the time of flow is too short to determine with satisfactory precision, or may not be determinable because the fiber-reinforced concrete flows freely through the inverted cone.

6.1.4 Air content when air-entrainment is required, ba on the air content of samples taken at the point of dischar or when appropriate the point of placement, subject to tolerances hereinafter specified;

6.1.4.1 Air-entrainment shall not be specified for sh crete placed by the dry process.

NOTE 6—In selecting the specified air content, the purchaser sho consider the exposure conditions to which the concrete will be subjec Air contents less than shown in Table 1 may not produce adeq resistance to freezing and thawing. Air contents higher than the le shown may reduce strength without contributing further to freeze-t resistance.

6.1.5 When structural lightweight concrete is specifi the purchaser shall specify the unit weight as wet weig air-dry weight, or oven-dry weight.

NOTE 7—The unit weight of freshly mixed lightweight concrete, is the only unit weight determinable at the time of delivery, is alv higher than the air-dry or oven-dry weight. Definitions of, and meth for determining or calculating air-dry and oven-dry weights of li weight concrete are covered in Method C 567.

6.1.6 One of the following Alternatives, 1, 2, or 3, shal used as the basis for determining the proportions of fiber-reinforced concrete or fiber-reinforced shotcrete of quality required.

6.2 *Alternative Number 1:*

6.2.1 When the purchaser assumes responsibility for m ture proportioning, the following parameters shall also specified by the purchaser:

6.2.1.1 The cement content in pounds per cubic yard kilograms per cubic metre),

6.2.1.2 If mineral admixtures are required, the type, amounts to be used in pounds per cubic yard (or kilogr per cubic metre), or in percentages by weight of cement.

6.2.1.3 The maximum allowable amount of mixing w in gallons per cubic yard or litres per cubic metre, incluc surface moisture on the aggregates, but excluding w absorbed by the aggregate,

6.2.1.4 If air-entraining admixtures are required, the t name, and dosage range to be used to achieve the speci air content, (see 6.1.4),

6.2.1.5 If chemical admixtures are required, the t name, and dosage range to be used, and:

6.2.1.6 The type of fibers to be used and the amour pounds per cubic yard (or kilograms per cubic metre), Classification Section).

NOTE 8—The dosage of air-entraining, water-reducing (incl high-range), accelerating, and retarding admixtures needed to satisf material performance requirements varies. Therefore, dosage r should be specified to ensure that the material performance rec ments can be met.

TABLE 1 Recommended Total Air Content for Air-Entrained Concrete[A,B]

Exposure Condition[C]	Total Air Content, %						
	Nominal Maximum Sizes of Aggregate, in. (mm)						
	⅜ (9.5)	½ (12.5)	¾ (19.0)	1 (25.0)	1½ (37.5)	2 (50.0)	3 (75.0
Mild	4.5	4.0	3.5	3.0	2.5	2.0	1.5
Moderate	6.0	5.5	5.0	4.5	4.5	4.0	3.5
Severe	7.5	7.0	6.0	6.0	5.5	5.0	4.5

[A] For air-entrained concrete, when specified.

[B] Unless exposure conditions dictate otherwise, air contents recommended above may be reduced by up to 1 % for concretes with specified compressive streng of 5000 psi (34.5 MPa) or above.

[C] For description of exposure conditions, refer to ACI 211.1, Table 5 3.3 with attention to accompanying footnotes.

NOTE 9—The purchaser, in selecting requirements for which he assumes responsibility should give consideration to requirements for workability, placeability, durability, surface texture, and density. The purchaser is referred to ACI Practices 211.1 and 211.2 for selecting proportions that will result in concrete suitable for various types of structures and conditions of exposure, and to ACI Report 544.3R for selecting concrete and fiber parameters suitable for fiber-reinforced concrete. For guidance on selecting proportions for fiber-reinforced shotcrete, the purchaser is referred to ACI Reports 506.1R and 506.R and ACI Specification 506.2.

6.2.2 At the request of the purchaser, the manufacturer shall, prior to the actual delivery of concrete, furnish a statement to the purchaser giving the sources, specific gravities, sieve analyses, and saturated surface-dry weights of fine and coarse aggregates, and the amount of mixing water per cubic yard or cubic metre that will be used in the manufacture of each class of concrete ordered by the purchaser.

6.3 *Alternative Number 2:*

6.3.1 When the purchaser requires the manufacturer to assume full responsibility for mixture proportioning (see Note 9), the purchaser shall also specify the following:

6.3.1.1 Requirements for flexural toughness, or first-crack strength, or both, determined in accordance with Method C 1018, or, at the option of the purchaser, for flexural strength determined in accordance with Method C 78, using samples obtained at the point of discharge, or when appropriate at the point of placement. At the option of the purchaser, compressive strength (Method C 39) shall be specified when the flexural requirements are considered inadequate for ensuring the quality of the matrix of the fiber-reinforced concrete. Unless accelerated curing and testing in accordance with the warm water or boiling water procedures of Method C 684 is specified, tests shall be performed after standard moist curing in accordance with Practices C 31 or C 192 at 28 days, or such other ages as are specified by the purchaser.

NOTE 10—The level of toughness achieved in any mixture is primarily a function of the type, length, and amount of fibers employed, so it is recommended that, when specifying requirements for flexural toughness, the requirements be stated in terms of one of the four levels of performance identified in the Performance Requirements section of this Specification.

NOTE 11—While first-crack strength is affected by the type and amount of fibers, it is more dependent on the characteristics of the mortar or concrete matrix, so it is recommended that the purchaser, when specifying first-crack strength, consider factors known to influence the strength of normal concrete such as, water-cement ratio, aggregate maximum size, and the presence of chemical or mineral admixtures.

6.3.2 At the request of the purchaser, the manufacturer shall, prior to the actual delivery of concrete, furnish a statement to the purchaser giving the sources, specific gravities, sieve analyses, and saturated surface-dry weights of fine and coarse aggregates, the dry weights of cement and mineral admixtures, the type, dimensions, and weight of fibers, the quantities, types and names of chemical and air-entraining admixtures (if any), and the amount of mixing water per cubic yard or cubic metre that will be used in the manufacture of each class of concrete ordered by the purchaser. The manufacturer shall also furnish evidence satisfactory to the purchaser that the materials to be used and the proportions selected will produce fiber-reinforced concrete or shotcrete of the quality specified.

6.4 *Alternative Number 3:*

6.4.1 When the purchaser requires the manufacturer to assume responsibility for mixture proportioning with the minimum allowable cement content specified (see Note 9), the purchaser shall also specify the following:

6.4.1.1 Requirements for flexural toughness, or first-crack strength, or both, determined in accordance with Method C 1018, or, at the option of the purchaser, for flexural strength determined in accordance with Method C 78, using samples obtained at the point of discharge, or when appropriate the point of placement. At the option of the purchaser, compressive strength (Method C 39) shall be specified when the flexural requirements are considered inadequate for ensuring the quality of the matrix of the fiber-reinforced concrete. Unless accelerated curing and testing in accordance with the warm water or boiling water procedures of Method C 684 is specified, tests shall be performed after standard moist curing in accordance with Practices C 31 or C 192 at 28 days, or such other ages as are specified by the purchaser (see Notes 10 and 11).

6.4.1.2 Minimum cement content in pounds per cubic yard (or kilograms per cubic metre).

6.4.1.3 If admixtures are required, the type, name, and dosage to be used. The cement content shall not be reduced when admixtures are used.

NOTE 12—Alternative Number 3 can be distinctive and useful only if the designated minimum cement content is at about the same level that would ordinarily be required for the mechanical properties, aggregate size, and workability specified. It must be an amount that will be sufficient to ensure durability under expected service conditions, as well as satisfactory surface texture and density. For additional information refer to ACI Practices 211.1 and 211.2.

6.4.2 At the request of the purchaser, the manufacturer shall, prior to the actual delivery of the concrete, furnish a statement to the purchaser giving the sources, specific gravities, sieve analyses and saturated surface-dry weights of fine and coarse aggregates, the dry weights of cement and mineral admixtures, the type, dimensions, and weight of fibers, the quantities, types and names of chemical and air-entraining admixtures (if any), and the amount of mixing water per cubic yard or cubic metre that will be used in the manufacture of each class of concrete ordered by the purchaser. The manufacturer shall also furnish evidence satisfactory to the purchaser that the materials to be used and the proportions selected will produce fiber-reinforced concrete or shotcrete of the quality specified.

6.5 The proportions arrived at by Alternatives 1, 2, or 3 for each class of fiber-reinforced concrete or shotcrete approved for use in a project shall be assigned a designation to facilitate identification of each mixture delivered to the project. A certified copy of the proportions of all mixtures as established in Alternatives 1, 2, and 3 shall be kept on file by the manufacturer.

7. Materials and Manufacture

7.1 In the absence of designated applicable specification covering requirements for quality of materials, the following specifications shall govern:

7.1.1 *Cement*—Cement shall conform to Specification C 150 or C 595.

7.1.2 *Aggregates*—Aggregates shall conform to Specifica tions C 33, C 330, or C 637 consistent with the type of concrete required.

7.1.3 *Water:*

7.1.3.1 The mixing water shall be clear and apparently clean. If it contains quantities of substances that discolor it or make it smell or taste unusual or objectionable or cause suspicion, it shall not be used unless service records of concrete made with it or other information indicates that it is not injurious to the quality of the concrete. Water of questionable quality shall be subject to the acceptance criteria of Table 2.

7.1.3.2 Wash water from mixer washout operations may be used as mixing water provided tests of wash water comply with the physical tests of Table 2. Wash water shall be tested at a weekly interval for approximately 4 weeks, and thereafter at a monthly interval provided that no single test exceeds the applicable limit. Optional chemical requirements in accordance with Table 3 may be specified by the purchaser when appropriate for the construction. The testing frequency for chemical limits shall be as given above unless otherwise specified by the purchaser.

NOTE 13—When recycled wash water is used, attention should be given to effects on the dosage rate and batching sequence of air-entraining and other chemical admixtures, and a uniform amount should be used in consecutive batches.

7.1.4 *Admixtures*—Admixtures for conventionally mixed fiber-reinforced concrete shall conform to Specifications C 260, C 618, C 494, or C 1017 whichever is applicable.

7.1.5 *Fibers*—Fibers shall be capable of producing fiber-reinforced concrete meeting the requirements of this specification. Steel fibers shall conform to Specification A 820.

8. Measuring Materials

8.1 Except as otherwise specifically permitted by the purchaser, cement, pozzolans, fine and coarse aggregates, mixing water, and admixtures shall be measured in accordance with the applicable requirements of Specification C 94 or C 685.

8.2 Fibers shall be measured by weight. When approved by the purchaser, fibers may be measured in bags, boxes, or like containers. Such bags, boxes, or containers shall be sealed by the fiber manufacturer and shall have the weight contained therein clearly marked. No fraction of an unsealed bag, box or like container delivered unsealed, or left over from previous work, shall be used unless weighed.

8.3 Prepackaged, dry, combined materials, including fibers, shall comply with the packaging and marking requirements of Specification C 387 and shall be accepted for use provided that after addition of water, the resulting fiber-reinforced concrete or shotcrete meets the performance requirements of this specification.

TABLE 2 Acceptance Criteria for Questionable Water Suppliers

	Limits	Test Method
Compressive strength, min % control at 7 days	90	C 109[A]
Time of set deviation from control, h: min	from 1:00 early to 1:30 later	C 191[A]

[A] Comparisons shall be based on fixed proportions and the same volume of test water compared to control mix using city water or distilled water.

TABLE 3 Chemical Limitations for Wash Water Used as Mixing Water

	Limits	Test Method[A]
Chemical requirements, maximum concentration in mixing water, ppm[B]		
Chloride as Cl, ppm:		D 512
Prestressed concrete or in bridge decks	500[C]	
Other reinforced concrete in most environments or containing aluminum embedments or dissimilar metals or with stay-in-place galvanized metal forms	1000[C]	
Sulfate as SO_4, ppm	3000	D 516
Alkalies as (Na_2O + 0.658 K_2O), ppm	600	
Total solids, ppm	50 000	AASHTO T2[illegible]

[A] Other test methods that have been demonstrated to yield comparable results may be used.

[B] Wash water reused as mixing water in concrete may exceed the listed concentrations of chloride and sulfate if it can be shown that the concentration calculated in the total mixing water, including mixing water on the aggregates and other sources does not exceed the stated limits.

[C] For conditions allowing use of $CaCl_2$ accelerator as an admixture, the chloride limitation may be waived by the purchaser.

9. Batching Plant

9.1 Batching plant used for the preparation of batch mixed fiber-reinforced concrete shall comply with the applicable requirements of Specification C 94.

NOTE 14—A vibrating screen or other device for separating fibers may be required to avoid clumping of some types of fibers prior to mixing with concrete.

10. Mixing Equipment

10.1 Mixers or agitators for batch-mixed fiber-reinforced concrete shall comply with the applicable requirements of Specification C 94.

10.2 Mixers for continuously mixed fiber-reinforced concrete shall comply with the applicable provisions of Specification C 685.

11. Mixing and Delivery

11.1 Batch-mixed fiber-reinforced concrete, whether prepared on site or at a location remote from the site, shall be mixed and delivered to the point designated by the purchaser in accordance with the applicable requirements of Specification C 94.

11.2 Continuously mixed fiber-reinforced concrete whether prepared on site or at a location remote from the site, shall be mixed and delivered to the point designated by the purchaser in accordance with the applicable requirements of Specification C 685.

11.3 Fiber-reinforced concrete shall be free of fiber balls when delivered.

12. Batch Ticket Information

12.1 The manufacturer of the fiber-reinforced concrete shall furnish to the purchaser a delivery ticket or statement of particulars on which is printed, stamped, or written, information in one of the following two alternative formats:

12.1.1 *Batch-Mixing Format*—The details identified in the applicable requirements of Specification C 94, and details of the type, brand, and amount of fibers used.

12.1.2 *Continuous Mixing Format*—The details identified in the applicable requirements of Specification C 685, and details of the type, brand, and amount of fibers used.

13. Inspection of Materials, Production, and Delivery

13.1 The manufacturer shall afford the inspector all reasonable access, without charge, for making necessary checks of the production facilities and for securing necessary samples to determine if the materials used in the fiber-reinforced concrete or shotcrete comply with the requirements of this specification. Inspection, sampling, and testing shall not interfere unnecessarily with the manufacturing and delivery operations.

14. Sampling

14.1 The contractor shall afford the inspector all reasonable access, without charge, for the procurement of samples of freshly mixed fiber-reinforced concrete or shotcrete at the time of placement to determine compliance with the requirements of this specification.

14.2 Samples of batch-mixed fiber-reinforced concrete shall be obtained in accordance with Method C 172, except that wet-sieving shall not be permitted. Sampling for uniformity tests shall be in accordance with Specification C 94.

14.3 Samples of continuously mixed fiber-reinforced concrete shall be obtained in accordance with the applicable requirements of Specification C 685, except that wet-sieving shall not be permitted. Sampling for uniformity tests shall be in accordance with Specification C 685.

15. Workability and Air Content Tests

15.1 Make tests for workability and air content at the time of placement at the option of the inspector as often as necessary for control checks and acceptance purposes, and always when specimens for tests on hardened concrete are made. When water is added in accordance with the requirements of this specification (see Tolerances in Workability Section), repeat all tests, and use the results of the second set of tests to establish whether or not the requirements of this specification are met.

15.2 If the measured slump, time of flow, or air content fall outside the limits permitted by this specification, make a check test immediately on another portion of the same sample. If the results again fall outside the permitted limits, the material represented by the sample fails to meet the requirements of this specification.

16. Tolerances in Workability

16.1 Unless other tolerances are included in the project specifications, the following shall apply to all forms of fiber-reinforced concrete except dry-mix shotcrete.

16.1.1 When the project specifications for slump are written as a "maximum" or "not to exceed" requirement:

	Specified Slump	
	If 3 in. (75 mm) or less	If more than 3 in. (75 mm)
Plus Tolerance	0	0
Minus Tolerance	1½ in. (40 mm)	2½ in. (65 mm)

When the project specifications for time of flow are written as a "minimum" or "not less than" requirement:

	Specified Time of Flow	
	If 15 s or less	If more than 15 s
Plus Tolerance	5 s	10 s
Minus Tolerance	0 s	0 s

These tolerances apply only if one addition of water is permitted on the job provided such addition does not increase the water-cement ratio above the maximum permitted by the project specifications.

NOTE 15—The slump of a fiber-reinforced concrete is less than the slump of an otherwise identical concrete without fibers. The magnitude of the difference depends strongly on the amount and type of fibers, so it is recommended that trial mixtures representing the amount and type of fibers to be used in the work be prepared and tested to ensure that the specified slump requirements are met. This recommendation is also appropriate when workability is specified in terms of time of flow.

16.1.2 When the project specifications for slump are not written as a "maximum" or "not to exceed" requirement:

Tolerances for Nominal Slumps

For Specified Slump of	Tolerance
2 in. (50 mm) and less	±½ in. (15 mm)
2 to 4 in. (50 to 100 mm)	±1 in. (25 mm)
more than 4 in. (100 mm)	±1½ in. (40 mm)

When the project specifications for time of flow are not written as a "minimum" or "not less than" requirement:

Tolerances for Time of Flow

For Specified Time of Flow of	Tolerance
8 to 15 s	±3 s
more than 15 s	±5 s

16.2 Fiber-reinforced concrete shall be available within the permissible range of slump or time of flow for a period of 30 min starting either on arrival at the job site or after the permitted slump adjustment, whichever is later. The first and last ¼ yd^3 or ¼ m^3 discharged are exempt from this requirement. If the user is unprepared for discharge of the material at the job site, the manufacturer shall not be responsible for failure to meet slump or time of flow requirements after 30 min have elapsed beyond either the actual arrival time at the job site or the requested delivery time, whichever is later.

17. Tolerance in Air Content

17.1 When air-entrainment is specified, the total air content measured using Method C 173 or Method C 231 shall be within a tolerance of ±1.5 of the specified value in percent.

18. Acceptance Testing of Hardened Fiber-Reinforced Concrete or Shotcrete

18.1 Obtain material for the preparation of test specimens in accordance with the sampling section of this specification.

18.2 When flexural toughness parameters, or first-crack strength, or both, are used as the basis for acceptance of fiber-reinforced concrete or shotcrete, make, condition, and test sets of test specimens in accordance with Method C 1018.

18.3 When flexural strength is used as the basis for acceptance, make sets of at least three test specimens in accordance with the requirements for sampling and conditioning given in Method C 1018, and test in accordance with the applicable requirements of Methods C 42 or C 78. Test specimens representing thin sections, as defined in Method C 1018, or specimens representing fiber-reinforced shotcrete of any thickness, shall be tested as cast or placed without being turned on their sides before placement on the support system. Acceptance shall not be based on flexural strength alone when toughness is important.

NOTE 16—Method C 1018 provides for the determination of flexural strength when required by the purchaser. For many type-amount fiber combinations, the flexural strength is not significantly greater than the first-crack strength.

18.4 When compressive strength is used as part of the basis for acceptance of fiber-reinforced concrete, make sets of at least two test specimens in accordance with the applicable requirements of Methods C 31 and C 192 and condition and test in accordance with Methods C 39 or C 42. Acceptance shall not be based on compressive strength alone.

18.5 The testing laboratory performing acceptance tests shall comply with the requirements of Practice C 1077.

19. Frequency of Tests

19.1 The frequency of tests on hardened fiber-reinforced concrete or shotcrete shall be in accordance with the following requirements:

19.1.1 *Batch-Mixing*—Tests shall be made with a frequency of not less than one test for each 150 yd^3 (115 m^3). Each test shall be made from a separate batch. On each day fiber-reinforced concrete is mixed, at least one test shall be made for each class of material.

19.1.1 *Continuous Mixing*—Tests shall be made for each 25 yd^3 (19 m^3) or fraction thereof, or whenever significant changes have been made in the proportioning controls. On each day fiber-reinforced concrete is mixed, at least one test shall be made for each class of material.

19.1.3 *Shotcrete*—Tests shall be made for each 50 yd^3 (38 m^3) placed using specimens sawed or cored from the structure or from corresponding test panels. On each day fiber-reinforced shotcrete is prepared, at least one test shall be made for each class of material.

19.2 The representative of the purchaser shall ascertain and record the delivery-ticket number or equivalent information and the exact location in the work at which the material represented by each test is deposited.

20. Calculation of Test Results

20.1 A test result shall be based on the mean of the property values for a set of hardened concrete test specimens constituting a test unit as defined herein or in the applicable test method.

20.2 Any individual test specimen in a set constituting a test unit, as defined herein or in the applicable test method, shall be deemed defective and discarded if it shows definite evidence of improper sampling, molding, handling, curing, or testing, and the mean of the property values for the remaining test specimens shall be considered the test result. If more than one specimen in the set is deemed defective on this basis, the test result shall be rejected.

21. Performance Requirements

21.1 Unless specifically excluded by the purchaser when ordering material in accordance with Alternatives Number 2 or 3, fiber-reinforced concrete or shotcrete prepared in accordance with this specification shall meet the following requirements:

21.2 For flexural toughness parameters defined in accordance with Method C 1018, the test results shall equal or exceed the specified values at the applicable test age.

NOTE 17—For the purchaser unfamiliar with the levels of perfo ance associated with various types and amounts of fibers, the follov is a guide on how and at what level performance should be specified various types of fiber:

Performance Level	Toughness Index, I_5 Specified Value	Toughness Index, I_5 Test Result	Toughness Index, I_{10} Specified Value	Toughness Index, I_{10} Test Result
I	2.7	3.0	5.4	6.0
II	3.6	4.0	7.2	8.0
III	4.5	5.0	9 0	10.0
IV	5.4	6.0	10.8	12.0

Due to variation in materials, mixing operations, fiber distribu and testing procedures, the average values of the toughness indice and I_{10}, represented by each test result must be greater than the speci values if a high proportion of the test results are to equal or exceed specified values. The relationship between the specified value and test result given above is based on the rationale described in Recommended Practice 214, specifically the condition that no n than 10 % of test results fall below the specified value, and on coefficients of variation of 12 % and 14 % for I_5 and I_{10} respecti given in the Precision and Bias section of Method C 1018.

Performance Level 4 is achievable only with high concentration steel fibers having deformed surfaces or end anchorages offering supe resistance to pullout from the cementitious matrix. Such concentrat may be difficult to disperse uniformly in concrete using conventi mixing procedures unless a high-range water-reducing admixtur employed. Performance Levels 2 and 3 can be achieved by a varie steel fibers in moderate concentrations readily amenable to conventi mixing. These performance levels may also be achieved using glass fi which are more amenable to shotcreting than conventional mi because of damage to the fibers by most conventional mixing proce Performance Levels 3 and 4 are not normally attainable using maximum amounts of polypropylene fibers which can be unifor dispersed in concrete using conventional mixing procedures.

NOTE 18—A toughness requirement should not be specified w fibers are used only to control plastic shrinkage cracking.

21.3 When first-crack strength, flexural strength, or co pressive strength are performance requirements, the results shall equal or exceed the specified values at applicable test age.

21.4 When the fiber-reinforced concrete is to be expo to cycles of freezing and thawing, and the purchaser requ evidence of satisfactory durability, such evidence shal provided by the manufacturer. A proven record of satis tory freeze-thaw durability for concrete with or with fibers, made using the same air content, aggregates, mixture proportions as the fiber-reinforced concrete sp fied for the work, shall be considered acceptable evide when the concrete has been in place for at least two win In the absence of such a record, satisfactory durability s be demonstrated for the fiber-reinforced concrete prop for the work by the attainment of an average durability fa of at least 80 % for a set of three specimens tested accor to Procedure A of Method C 666.

22. Failure to Meet Requirements

22.1 When fiber-reinforced concrete or shotcrete fail meet the requirements of this specification, the manufact and the purchaser shall confer to determine whether ag ment can be reached as to what adjustment, if any sha made. If agreement on a mutually satisfactory adjustn cannot be reached by the manufacturer and the purchas decision shall be made by a panel of three qualified e neers, one of whom shall be designated by the purchaser, by the manufacturer, and the third chosen by these members of the panel. The question of responsibility for the cost of such arbitration shall be determined by the panel. Its decision shall be binding, except as modified by a court decision.

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ORIGINAL

Designation: C 1018 – 89[ε1]

Standard Test Method for Flexural Toughness and First-Crack Strength of Fiber-Reinforced Concrete (Using Beam With Third-Point Loading)[1]

This standard is issued under the fixed designation C 1018; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

[ε1] NOTE—Fig. 1 was corrected editorially in May 1990.

1. Scope

1.1 This test method evaluates the flexural toughness of fiber-reinforced concrete in terms of areas under the load-deflection curve obtained by testing a simply supported beam under third-point loading.

NOTE 1—Toughness determined in terms of areas under the load-deflection curve is an indication of the energy absorption capability of the particular test specimen, and, consequently, its magnitude depends directly on the geometrical characteristics of the test specimen and the loading system.

1.2 This test method provides for the determination of a number of ratios that serve as toughness indices which identify the pattern of material behavior up to the selected deflection criteria. These indices are determined by dividing the area under the load-deflection curve up to a specified deflection criterion, by the area up to the deflection at which first crack is deemed to have occurred.

NOTE 2—Index values may be increased by preferential alignment of fibers parallel to the longitudinal axis of the beam caused by fiber contact with the mold surfaces or by external vibration. However, index values appear to be independent of geometrical specimen and testing variables, such as span length, which do not directly affect fiber alignment.

1.3 This test method provides for the determination of the first-crack flexural strength using the load corresponding to the point on the load-deflection curve defined in 3.1.1 as first crack, and the formula for modulus of rupture given in Test Method C 78.

1.4 Values of flexural toughness and first-crack flexural strength stated in inch-pound units are to be regarded as the standard. Values of toughness indices are the same in all systems of units because the indices are ratios (see 1.2).

1.5 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

C 31 Practice for Making and Curing Concrete Test Specimens in the Field[2]

C 42 Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete[2]

C 78 Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)[2]

C 172 Method of Sampling Freshly Mixed Concrete[2]

C 192 Practice for Making and Curing Concrete Test Specimens in the Laboratory[2]

C 670 Practice for Preparing Precision and Bias Statements for Test Methods for Construction Materials[2]

C 823 Practice for Examination and Sampling of Hardened Concrete in Constructions[2]

E 4 Practices for Load Verification of Testing Machines[2]

3. Terminology

3.1 *Descriptions of Terms Specific to This Standard:*

3.1.1 *first crack*—the point on the load-deflection curve at which the form of the curve first becomes nonlinear (approximates the onset of cracking in the concrete matrix).

3.1.2 *first-crack deflection*—the deflection value on the load-deflection curve at first crack.

3.1.3 *first-crack strength*—the stress obtained when the load corresponding to first crack is inserted in the formula for modulus of rupture given in Test Method C 78.

3.1.4 *first-crack toughness*—the energy equivalent to the area under the load-deflection curve up to the first-crack deflection.

3.1.5 *toughness*—the energy equivalent to the area under the load-deflection curve up to a specified deflection.

3.1.6 *toughness indices*—the numbers obtained by dividing the area up to a specified deflection by the area up to first crack.

NOTE—Values of 5.0, 10.0, and 20.0 for I_5, I_{10}, and I_{20} respectively, as defined below, correspond to linear elastic material behavior up to first crack and perfectly plastic behavior thereafter (see Appendix X1).

3.1.6.1 *toughness index I_5*—the number obtained by dividing the area up to a deflection of 3.0 times the first-crack deflection by the area up to first crack.

3.1.6.2 *toughness index I_{10}*—the number obtained by

[1] This test method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03.04 on Fiber-Reinforced Concrete.

Current edition approved April 14, 1989. Published July 1989. Originally published as C 1018 – 84. Last previous edition C 1018 – 85.

[2] *Annual Book of ASTM Standards*, Vol 04.02.

dividing the area up to a deflection of 5.5 times the fi rack deflection by the area up to first crack.

3.1.6.3 *toughness index* I_{20}—the number obtained by dividing the area up to a deflection of 10.5 times the first-crack deflection by the area up to first crack.

3.1.6.4 *residual strength factor* $R_{5,10}$—the number obtained by calculating the value of 20 $(I_{10} - I_5)$.

3.1.6.5 *residual strength factor* $R_{10,20}$—the number obtained by calculating the value of 10 $(I_{20} - I_{10})$.

4. Summary of Test Method

4.1 Molded or sawn beams of fiber-reinforced concrete are tested in flexure using the third-point loading arrangement specified in Test Method C 78. Load and beam deflection are monitored either continuously by means of an X-Y plotter, or incrementally by means of dial gages read at sufficiently frequent intervals to ensure accurate reproduction of the load-deflection curve. A point termed first crack which corresponds approximately to the onset of cracking in the concrete matrix is identified on the load deflection curve. The first-crack load and deflection are used to determine the fi rack flexural strength and to establish end-point deflections for toughness calculations. Computations of toughness and toughness indices are based on areas under the load-deflection curve up to the first-crack deflection and up to the specified end-point deflection.

5. Significance and Use

5.1 The first-crack strength characterizes the behavior of the fiber-reinforced concrete up to the onset of cracking in the matrix, while the toughness indices characterize the toughness thereafter up to specified end-point deflections. Residual strength factors, which are derived directly from toughness indices, characterize the level of strength retained after first crack simply by expressing the average post-crack load over a specific deflection interval as a percentage of the load at first crack. The importance of each depends on the nature of the proposed application and the level of serviceability required in terms of cracking and deflection. Toughness and first-crack strength are influenced in different ways by the amount and type of fiber in the concrete matrix. In some cases, fibers may greatly increase the toughness, toughn indices, and residual strength factors determined by this test method while producing a first-crack strength only slightly greater than the flexural strength of the plain concrete matrix. In other cases, fibers may significantly increase the first-crack strength with only relatively small increases in toughness, toughness indices, and residual strength factors.

5.2 The toughness indices determined by this test method reflect the behavior of fiber-reinforced concrete under static flexural loading. The absolute values of toughness determined to compute the toughness indices are of little practical significance since they are directly dependent upon geometrical variables associated with the specimen and the loading arrangement.

NOTE 3—In applications where the energy absorption capability of a structural concrete element is important, it may be possible to obtain some indication of its performance by testing a specimen equivalent to the element in terms of size, span, and mode of loading.

5.3 In determining which toughness index is most appropriate as a measure of material performance for a specific application, the level of serviceability required in term cracking and deflection shall be considered, and an ic appropriate to the service conditions shall be selectec accordance with the rationale described in 9.6 and Appendix X1.

5.4 Values of toughness indices, residual strength fac and first-crack strength may be used for comparing performance of various fiber-reinforced concretes during mixture proportioning process or in research and deve ment work. They may also be used to monitor conc quality, to verify compliance with construction speci tions, or to evaluate the quality of concrete already in ser

5.5 Values of toughness indices, residual strength fac and first-crack strength obtained using the 14 by 4 by (350 by 100 by 100 mm) preferred standard size of mo specimen may not necessarily correspond with the perfc ance of larger or smaller molded specimens, concrete in l structural units, or specimens sawn from such units, bec of differences in the degree of preferential fiber alignr parallel to the longitudinal axis of the specimen.

5.5.1 Preferential fiber alignment is likely to occu molded specimens when fibers in the vicinity of the r surfaces tend to align in the plane of the surface, and is pronounced in specimens of small cross-section contai long fibers.

5.5.2 In thin concrete sections, such as overlays shotcrete linings, fibers tend to align in the plane o section, so in-place performance is best evaluated either molded or sawn specimens of depth equal tc thickness of the section. Consequently, toughness inc residual strength values, and first-crack strengths for sections may differ from those for standard molded s mens of nominally identical concrete.

5.5.3 External vibration promotes preferential align of fibers parallel to the vibrating surface of the forr screeding device used, while internal vibration does not this effect. Consequently, toughness indices, residual stre values, and first-crack strengths for identical concrete s mens prepared using the two kinds of vibration may di

5.5.4 Preferential fiber alignment is negligible in concrete because the aligning effect of mold surfac absent and because internal vibration is often use toughness indices, residual strength values, and first- strengths for standard molded specimens may differ those for sawn specimens of nominally identical concr

6. Apparatus

6.1 *Testing Machine*—The testing machine shall accordance with Test Method C 78 and shall, in additic capable of operating in a manner which produces a trolled and constant rate of increase of deflection c specimen. A testing machine capable only of produc constant rate of increase of load is not suitable for lishing the load-deflection curve after the maximum loa been reached.

6.2 *Deflection-Measuring Equipment*—Devices suc electronic transducers or mechanical dial gages sha located either at both loading points, or at the mid-sp accurately determine the net deflection of the test spec under load exclusive of any effects due to seating or tw of the specimen on its supports. Determination of a deflection at the loading points is preferable if the energy absorption capability of the test specimen is to be accurately assessed.

6.3 *X-Y Plotter*—An X-Y plotter coupled directly to electronic outputs of load and deflection is the preferred means of expediently and accurately obtaining the load-deflection curve. When deflection is measured at the two loading points, the average deflection shall be plotted. If an X-Y plotter is not available, incremental tabulations of load and deflection may be used to manually plot the curve.

NOTE 4—Accurate determination of the areas under the load-deflection curve subsequently needed for computation of toughness indices is only possible when the scales initially chosen for load and deflection are reasonably large. A load scale on which 1 in. (25 mm) corresponds to a flexural stress of the order of 150 psi (1 MPa) is recommended. For the preferred 14 by 4 by 4 in. (350 by 100 by 100 mm) specimen size, a deflection scale on which 1 in. (25 mm) corresponds to a deflection of the order of 0.004 in. (0.1 mm) is recommended for testing up to the I_{10} deflection criterion. When testing is continued to a higher end-point deflection, the scale may have to be reduced to avoid excessively large load-deflection plots. With some plotting equipment it is possible to use a relatively large scale up to the I_{10} criterion and switch to a smaller scale at higher deflections without interrupting the test. This keeps the size of the plot reasonable without adversely affecting the ability to accurately determine the area up to first crack and the areas up to the I_5 and I_{10} deflection criteria.

7. Sampling, Test Specimens, and Test Units

7.1 *General Requirements*—The nominal maximum size of aggregate and cross-sectional dimensions of test specimens shall be in accordance with Practice C 31 or Practice C 192 when using molded specimens, or in accordance with Method C 42 when using sawn specimens, except when the following specific requirements are contravened:

7.1.1 The length of test specimens shall be at least 2 in. (50 mm) greater than three times the depth, and in any case not less than 14 in. (350 mm).

7.1.2 The width of test specimens shall be at least three times the maximum fiber length.

7.1.3 The depth and size of test specimens shall conform to either of the following two sets of requirements:

7.1.3.1 *Thick Sections*—The depth of test specimens shall be at least three times the maximum fiber length. Subject to meeting this requirement and the requirements of 7.1, 7.1.1, and 7.1.2, the preferred specimen size is 14 by 4 by 4 in. (350 by 100 by 100 mm). When the preferred size is not large enough to meet all of these requirements, specimens of square cross-section large enough to meet the requirements shall be tested.

7.1.3.2 *Thin Sections*—When the requirements of 7.1 and 7.1.3.1 are not met in the application in which the concrete is to be used, as for example in overlays or shotcrete linings, specimens of depth equal to the section thickness actually used shall be tested.

NOTE 5—When testing freshly mixed fiber-reinforced concrete, it may be desirable to prepare additional specimens of the preferred standard size in order to make proper comparisons of their performance with results obtained on other jobs or reported in the literature.

7.2 *Freshly Mixed Concrete*—Samples of freshly mixed fiber-reinforced concrete for the preparation of test specimens shall be obtained in accordance with Method C 172.

7.2.1 Specimens shall be molded in accordance with Practice C 31 or Practice C 192, except that compaction shall be by external vibration, as internal vibration or rodding may produce nonuniform fiber distribution. Make sure that the time of vibration is sufficient to ensure adequate consolidation, as fiber-reinforced concrete requires a longer vibration time than concrete not containing fibers, especially when the fiber concentration is relatively high. Take care to avoid placing the concrete in a manner which produces lack of fiber continuity between successive placements by using a wide shovel or scoop and placing each lift of concrete uniformly along the length of the mold. Use a single layer for specimens of depth 3 in. (75 mm) or less and two layers for specimens of depth greater than 3 in. (75 mm).

7.2.2 In placing the final layer, attempt to add an amount of concrete that will exactly fill the mold after compaction. When trowelling the top surface, continue vibration in order to ensure that fibers do not protrude from the finished surface.

7.2.3 Curing shall be in accordance with Practice C 31 or Practice C 192.

7.3 *Hardened Concrete*—Samples of hardened fiber-reinforced concrete from structures shall be selected in accordance with Practice C 823.

7.3.1 Sawn specimens shall be prepared and cured in accordance with Method C 42.

7.4 *Test Unit*—At least three specimens from each sample of fresh or hardened concrete shall be prepared for testing.

8. Conditioning

8.1 From the time test specimens are removed from their curing environment until testing is completed, drying shall be minimized by applying a curing compound or by other appropriate techniques.

9. Procedure

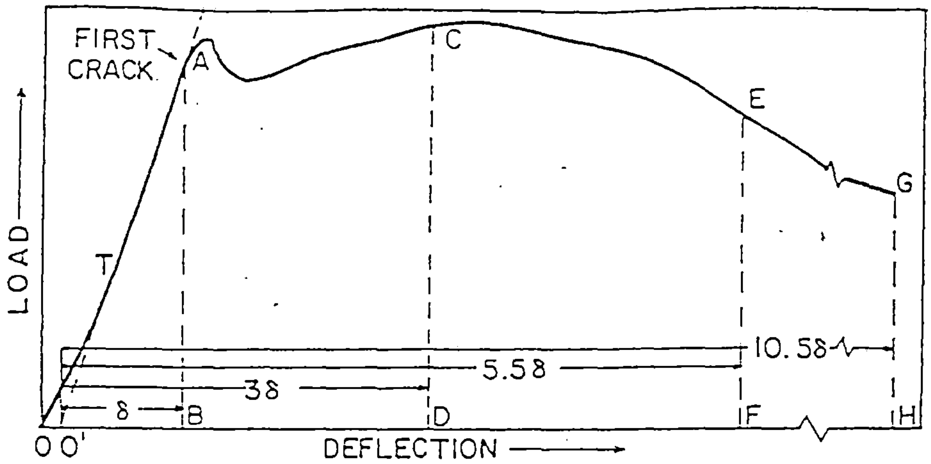
9.1 Molded or sawn specimens representing thick sections, as defined in 7.1.3.1, shall be turned on their side with respect to the position as cast before placing on the support system. Molded or sawn specimens representing thin sections, as defined in 7.1.3.2, shall be tested as cast without turning. Specimens representing shotcrete panels of any thickness shall be tested as placed without turning.

9.2 Arrange the specimen and the loading system so that the specimen is loaded at the third points in accordance with Test Method C 78. The span length shall be three times the specimen depth or 12 in. (300 mm), whichever is greater. If before loading, full contact is not obtained between the specimen, the load-applying devices, and the supports, grind or cap the contact surfaces of the specimen, or shim with strips of leather in accordance with Test Method C 78.

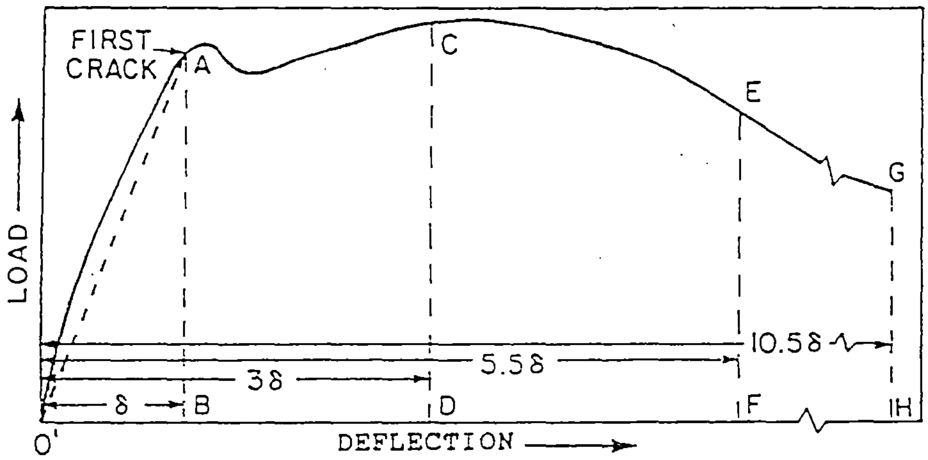
9.3 Operate the testing machine so that the deflection of the specimen at the mid-span increases at a constant rate within the range 0.002 to 0.004 in./min (0.05 to 0.10 mm/min) until the specified end-point deflection is reached. Use 0.87 times these limits when deflection is measured at the loading points.

NOTE 6—Testing machines capable of automatically controlling the rate of movement of the loading heads are well suited but not essential to this procedure.

9.4 Exercise care to ensure that the measured deflections are the net values exclusive of any extraneous effects due to seating or twisting of the specimen on its supports or



(a) Concave upwards to first crack



(b) Convex upwards to first crack

FIG. 1 Important Characteristics of the Load-Deflection Curve

deformation of the support system. An upwards concavity in the initial portion of the load-deflection curve is indicative of these extraneous effects, and shall be corrected as shown in 10.1, which is based on the assumption that the initial portion of the curve is linear.

NOTE 7—Location of deflection-measuring devices at the mid-width of the specimen minimizes the effect of twisting and reduces the number of devices needed to determine the average net deflection at the loading points or the mid-span. Location of additional deflection-measuring devices at the supports counters the effects of seating of the specimen on its supports and deformation of the support system, but increases the number of devices needed, and makes the processing of the data to obtain average net deflection more complex. Nominal deflections based only on measurements at the mid-span may be considerably larger than the corresponding net mid-span deflections obtained by subtracting the average of the deflections measured at the two supports from the corresponding nominal deflection at the mid-span. Toughness indices based on nominal mid-span deflections may be less than the equivalents calculated using net mid-span deflections. The magnitude of the difference may depend on the stiffness of the support system.

9.5 Unless otherwise specified by the purchaser, terminate the test at a deflection large enough to ensure that the area up to the end-point deflection of 5.5 times the first-crack deflection specified for the I_{10} index can be determined.

NOTE 8—For 14 by 4 by 4 in. (350 by 100 by 100 mm) specimer the first crack deflection is usually in the range 0.004 to 0.006 in. (0. to 0.15 mm) when based on mid-span measurement only. The corr sponding net mid-span deflection at first crack, that is the measur mid-span deflection minus the average deflection measured at t supports, is usually in the range 0.0015 to 0.0025 in. (0.038 to 0.0(mm).

9.6 When the level of serviceability appropriate to t particular application in terms of permissible deflection ar cracking indicates that the specified end-point deflectic should be higher, further testing to an appropriate deflectic criterion shall be specified at the option of the purchase Rationale for selection of end-point deflection is given i X1.3 of Appendix X1.

9.7 Make two measurements of the specimen depth ar width adjacent to the fracture (one at each face) to t nearest 0.05 in. (1.0 mm) to determine the average depth ar width.

9.8 Determine the position of the fracture by measurir the distance along the middle of the tension face from t fracture to the nearest end of the specimen.

NOTE 12—These levels of precision are based on data from a small number of investigations[3,4] conducted by experienced operators using good, but not necessarily the best possible equipment. The levels of precision achievable probably depend on the nature of the equipment used to produce the load-deflection curve, and the care exercised in computing the areas under this curve. As more sophisticated deflection-measuring and plotting devices become available, it may be possible to achieve 1S % values lower than those indicated.

[3] Johnston, C. D., "Precision of Flexural Strength and Toughness Parameters for Steel Fiber Reinforced Concrete," Cement, Concrete, and Aggregates, CCAGDP, Vol 4, No. 2, Winter 1982, pp 61–67.

[4] Unpublished data supplied by C. H. Henager.

12.2 No data are yet available to indicate whether the levels of precision for concretes containing other types of fibers, such as glass or polypropylene, differ from those quoted in 12.1.

12.3 *Multilaboratory Precision*—No data suitable for the evaluation of multilaboratory precision are yet available.

12.4 *Bias*—This test method has no bias since the properties determined can only be defined in terms of this test method.

APPENDIX

(Nonmandatory Information)

X1. RATIONALE FOR THE METHOD

X1.1 Absolute values of toughness up to the first-crack or other specified deflections depend entirely on geometrical variables associated with the specimen and the testing arrangement, and bear no direct relationship to the energy absorption capability of a structural element made with a fibrous concrete identical to that used to prepare specimens for testing according to this test method.

X1.2 Toughness indices I_5, I_{10}, and I_{20} enable actual performance to be compared with a readily understood reference level of performance. In this regard, values of 5.0, 10.0 and 20.0 for I_5, I_{10}, and I_{20} correspond to linear elastic material behavior up to first crack and perfectly plastic behavior thereafter[5] (Fig. X1.1). Such behavior is desirable for many applications requiring high toughness, and can be reached or exceeded only by careful selection of fiber type, fiber concentration, and concrete matrix parameters. The indices have the same meaning regardless of the cross-sectional size and span of the test specimen.

X1.3 When the conditions of serviceability or the purchaser's needs require a specified end-point deflection higher than that identified in 9.5, it is recommended that the end-point deflection be specified as a multiple of the first-crack deflection and that it be consistent with the rationale in X1.2. For example, an end-point deflection of 10.5 times the first-crack deflection permits calculation of the I_{20} index.

X1.4 The residual strength factors $R_{5,10}$ and $R_{10,20}$ represent the average level of strength retained after first crack as a percentage of the first-crack strength for the deflection intervals *CE* and *EG* respectively in Fig. 1 (a). Values of 100 correspond to perfectly plastic behavior (Fig. X1.1). Lower values indicate inferior performance. Plain concrete has residual strength factors of zero.

[5] Johnston, C. D., "Definition and Measurement of Toughness Parameters for Fiber-Reinforced Concrete," *Cement, Concrete, and Aggregates*, CCAGDP, Vol 4, No. 2, Winter 1982, pp 53–60.

I_{30} (or even I_{50}) would give a more accurate picture of toughness of a steel fibre reinforced concrete (load deflection curve)

9.9 When the fracture occurs outside the middle third of the span by more than 5 % of the span length, discard the results.

10. Calculation

10.1 If the load-deflection curve is slightly concave upwards throughout its initial portion, determine first crack by placing a straightedge coincident with that portion of the load-deflection curve which is essentially linear, and identifying the point at which the curvature first increases sharply and the slope of the curve exhibits a definite change, as at point *A* in Fig. 1(a). To correct for the extraneous effects identified in 9.4, extend the straight line, *AT*, representing the linear portion of the load-deflection curve from the point, *T*, at which it departs from the experimental curve to a new origin at point *O'*, as shown in Fig. 1(a). The line *O'TA* in Fig. 1(a) is used in subsequent area computations rather than the curve *OTA*.

If the load-deflection curve is slightly convex upwards throughout its initial portion, that is like the stress-strain curve for plain concrete in tension or compression, first crack is the point at which the curvature first increases sharply and the slope of the curve exhibits a definite change, as at A in Fig. 1 (b). The straight line O'A in Fig. 1(b) is used in subsequent area computations rather than the O'A portion of the curve.

10.2 Calculate the first-crack strength using the load corresponding to first crack on the load-deflection curve and the formula for modulus of rupture given in Test Method C 78.

NOTE 9—When the flexural strength is required, it may be determined using the maximum load attained on the load-deflection curve and the formula for modulus of rupture given in Test Method C 78. The value thus obtained may differ from the flexural strength obtained using the constant-rate-of-loading procedure specified in Test Method C 78.

10.3 Determine the first-crack deflection as the deflection corresponding to the length *O'B* in Fig. 1.

10.4 Determine the area under the load-deflection curve up to the first-crack deflection. This is the triangular area corresponding to *O'AB* in Fig. 1. If required, calculate the corresponding first-crack toughness in inch-pound or SI units.

10.5 Determine the area under the load-deflection curve up to a deflection of 3.0 times the first-crack deflection. This corresponds to the area *O'ACD* in Fig. 1 where *O'D* equals 3.0 times the first-crack deflection. Divide this area by the area up to first crack, obtained in accordance with 10.4, and report the number rounded to the nearest 0.1 as the toughness index I_5.

NOTE 10—Determination of the irregularly shaped areas needed to implement the instructions of this and subsequent sections 10.6 to 10.9 requires a planimeter, or application of Simpson's rule, or the counting of squares or other suitable elements of known area. When different deflection scales ar[illegible] on the same plot, [illegible] taken to ensure that this is taken [illegible] when co[illegible] area measurements to toughn[illegible].

10.6 Determine the area under the load-deflection curve up to a deflection of 5.5 times the first-crack deflection (area *O'*AEF in Fig. 1). Divide it by the area up to first crack, and report the number rounded to the nearest 0.1 as the toughness index I_{10}.

10.7 When required, determine the area under the load-deflection curve up to a deflection of 10.5 times the first-crack deflection (area *O'AGH* in Fig. 1). Divide it by the area up to first crack, and report the number rounded to the nearest 0.1 as the toughness index I_{20}.

10.8 Determine the residual strength factor $R_{5,10}$ as $20(I_{10} - I_5)$, and, when required, the residual strength factor $R_{10,20}$ as $10(I_{20} - I_{10})$. $R_{10/30} = 5(I_{30} - I_{10})$

NOTE 11—While the foregoing calculations presume that the load-deflection curve is determined in graphical form, it is not inconceivable that electronic equipment capable of digitally recording load and deflection may be developed, and that the recorded data may be analyzed by computer to determine relevant areas and toughness indices.

11. Report

11.1 The report shall include the following:

11.1.1 Type of specimen (molded or sawn) and specimen identification numbers or symbols,

11.1.2 Average width of specimen to the nearest 0.05 in. (1.0 mm),

11.1.3 Average depth of specimen to the nearest 0.05 in. (1.0 mm),

11.1.4 Span length to the nearest 0.1 in. (2.0 mm),

11.1.5 First-crack load and, when required, the maximum load, lbf(N),

11.1.6 First-crack deflection, in. (mm) to the nearest 0.0001 in. (0.002 mm), and the location where deflection was measured (mid-span or loading points),

11.1.7 First-crack strength and, when required, flexural strength to the nearest 5 psi (0.05 MPa),

11.1.8 First-crack toughness, lbf·in. (N·m), to the nearest 0.1 lbf·in. (0.01 N·m), when required,

11.1.9 Toughness indices I_5 and I_{10}, and the residual strength factor $R_{5,10}$,

11.1.10 Toughness index I_{20} and the residual strength factor $R_{10,20}$ when required,

11.1.11 Age of specimens at test,

11.1.12 Curing history and moisture condition of specimens at test,

11.1.13 Whether specimen was capped, ground, or shimmed, and

11.1.14 Defects in specimen prior to test and abnormalities in specimen behavior during test.

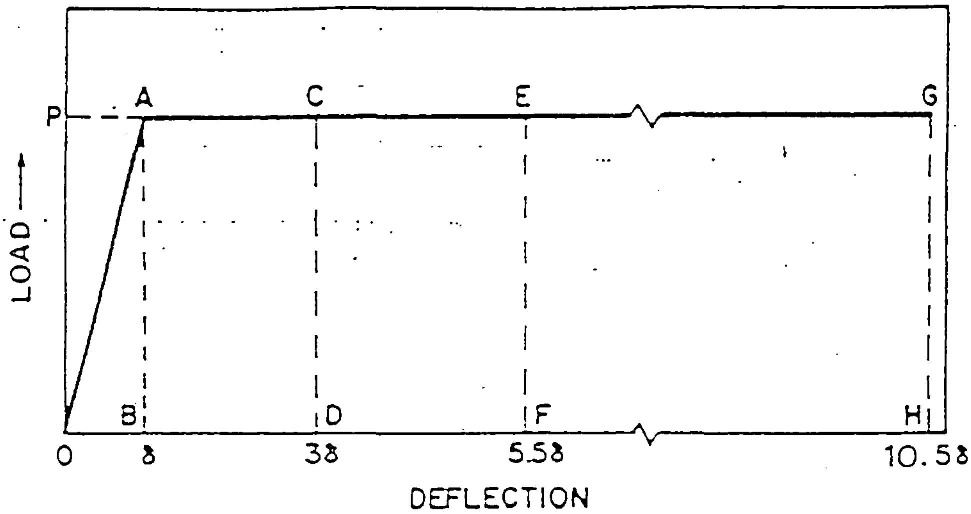
12. Precision and Bias

12.1 *Within-Laboratory Precision*—Single-operator values of the one-sigma limit in percent (1S %), defined in accordance with Practice C 670, have been determined for concretes containing steel fibers as follows:

Parameter	Within-Batch 1S %	Overall 1S % [A]
First-crack strength	5	7
First-crack toughness	10	12
Toughness index I_5	12	13
Toughness index I_{10}	14	16
Toughness index I_{20}	16	20
Flexural strength	5 to 8 [B]	8 to 10 [B]

[A] Inclusive of batch-to-batch variability, but not variability due to changes in specimen geometry, test span, and mode of loading.

[B] Upper limit appears applicable to relatively high fiber concentrations, 200 lb/yd³ (120 kg/m³) or more of straight uniform fibers, or 70 lb/yd³ (42 kg/m³) or more of deformed fibers.



Area Basis[A]	Index Designation	Deflection Criterion	Values of Toughness Indices		
			Plain Concrete	Elastic-Plastic Material	Observed Range for Fibrous Concrete
OACD	I_5	3δ	1.0	5.0	1 to 6
OAEF	I_{10}	5.5δ	1.0	10.0	1 to 12
OAGH	I_{20}	10.5δ	1.0	20.0	1 to 25

[A] Indices calculated by dividing this area by the area to the first crack OAB.

FIG. X1.1 Definition of Toughness Indices in Terms of Multiples of First-Crack Deflection and Elastic-Plastic Material Behavior

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TELEFAX

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THIS MESSAGE CONTAINS 1 PAGES (including this page)	

Dear Mr. Vervaeke,

The Foundation for Scientific and Industrial Research at the Norwegian Institute of Technology (SINTEF) is a research organization with approx. 2000 employees, and covers a range of disciplinary areas within technology. The reports are mostly restricted, including those for clients producing steel fibres.

A short state-of-the-art review will be:

1) About 330 km of accessible Norwegian hydro tunnels (from a total of 3500 km) are inspected in a project on long term stability. All damages and defects on support installations are reported. No corrosion problems on steel fibres in shotcrete lining are observed, only corrosion on the free fibre ends in the water stream. However, the most important stability problems are related to faults containing swelling clay in connection with the use of any kind of shotcrete, including 25 cm mesh reinforced shotcrete at zones with extreme swelling pressure.

2) Undersea road tunnels in Norway are investigated. No corrosion of steel fibres inside the shotcrete has been identified in spite of a relatively high chloride content. (A general paper from SINTEF in "Tunnels & Tunnelling", September 1988).

3) Special analyses of steel fibres in shotcrete are carried out. In this review we may say that the results are positive for the clients.

SINTEF Rock and Mineral Engineering will be pleased to be at your service.

Yours sincerely

Alf Thidemann

lf Thidemann
Research Manager

Undersea tunnels in Norway: a state-of-the-art review

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Portal of the Ellingsøy undersea road tunnel (No. 7 in Fig 1 and 7.2 in Fig 2).

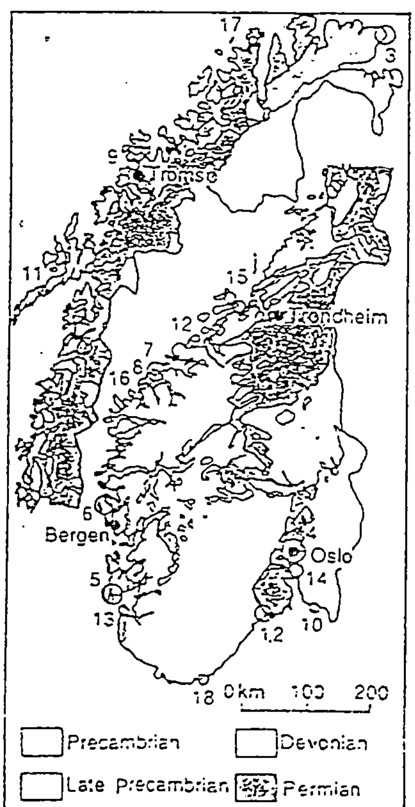


Fig 1. Completed subsea tunnels in Norway (1-7), those under construction (8-10) and those planned (11-18).

The shore line of Norway is characterised by a large number of fjords and straits and the majority of the population lives on the coast. Rock conditions in Scandinavia are generally good, and it is therefore logical that rock tunnels across fjords and straits have become increasingly popular for communication and industrial purposes.

Ten major under sea rock tunnels have been completed in Norway since 1974. A boom in subsea tunnelling, especially road tunnels, has been triggered by the successful completion of these projects. Some of these tunnels have previously been discussed in *Tunnels & Tunnelling*[1-5].

Experience gained from completed projects forms an important basis for the planning of new subsea tunnel projects. A research project was carried out at SINTEF during 1987 to summarise and evaluate the experience gained from Norwegian subsea tunnels. The research programme will be briefly presented here and some of the main results will be discussed.

Summary of tunnels

The state-of-the-art review has considered all the ten major subsea tunnels completed in Norway so far. The majority of these tunnels are related to fjord and strait crossings on the west coast, or are road tunnels or tunnels for oil and gas pipelines (Fig 1 and Table 1).

The total subsea length constructed to date is about 20km, and the greatest depth below sea level is about 250m. Longitudinal sections along the respective tunnels are shown in Fig 2.

For the most recent oil and gas pipeline tunnels the cross-sectional area is about 25m². For two-lane road tunnels the area is about 50m², and for three-lane road tunnels about 70m².

The Norwegian undersea tunnels are situated in a variety of geological structures ranging from typical hard rock such as Precambrian gneiss to less competent phyllite and poor quality schists and shales. All tunnels cross significant zones of weakness under the sea.

Three more undersea tunnels are under construction at present in Norway, and more than 30 are being planned or are under consideration. The majority of these are two- and three-lane road tunnels. Fig 1 indicates the locations of some of the new projects which are most likely to be constructed in the near future.

Research programme

The state-of-the-art study has concentrated in the main on the following topics:
- □ Preinvestigations
- □ Tunnelling results
- □ Behaviour of rock support materials

Among other points, the study includes discussions on the usefulness of the preinvestigations in predicting tunnelling conditions and a study into the effect of saline environments on the condition of rock support materials.

Owners, contractors and consultants for the tunnels which are documented have actively contributed to the study so that a broad overview has been possible. A total of ten technical reports have been published. These reports (in Norwegian) may be ordered from SINTEF.

Preinvestigations

The following preinvestigations are normally used in Norway for subsea tunnels:
1. Review of existing information.
2. Geological engineering field mapping.
3. Acoustic profiling.
4. Refraction seismic profiling.
5. Drilling.

Since most of a subsea tunnel route is normally covered by sea, the planning of such projects is to a great extent based on data from seismic investigations and drilling (Stages 3-5). The planning and performance of such investigations are discussed in detail by Beitnes & Blindheim[1], and Paulsson[7].

The total profile length of acoustic surveying is normally between 20 and 50 times the subsea length of the tunnel, while the total length of refraction seismic profiles is normally between 1.5 and 3.5 times the subsea length. Core drilling is most commonly carried out as inclined holes from the shore and under the sea, and with a drilled length of 30-40% of the subsea length.

Table 1. Norwegian rock tunnels beneath the sea floor completed prior to 1988.

	Project	Year completed	Bedrock	Cross-section	Total length	Subsea length	Lowest level
1.	Vollsfjord, water supply tunnel	1977	Precambrian gneiss	16m²	9.4km	0.6km	- 80m
2.	Frierfjord, gas pipe tunnel	1977	Precambrian gneiss/Cambro-Silurian sediments	16m²	3.6km	3.1km	- 252m
3.	Vardø, road tunnel	1981	Late Precambrian sandstone/shale	53m²	2.6km	1.7km	- 88m
4.	SRV — Slemmestad, sewer tunnel	1982	Cambro-Silurian shale/limestone	10m²	0.9km	0.7km	- 95m
5.	Karmøy — Kårstø, gas pipe tunnel						
	5.1 Karmsundet	1983	Caledonian greenstone and phyllite/ Precambrian gneiss	27m²	4.8km	2.1km	-180m
	5.2 Førdesfjord	1983	Precambrian gneiss	27m²	3.4km	1.6km	-170m
	5.3 Førlandsfjord	1983	Precambrian gneiss/Caledonian phyllite	27m²	3.9km	1.0km	-160m
6.	Hjartøy, oil pipe tunnel	1986	Precambrian gneiss	26m²	2.3km	1.8km	-105m
7.	Ålesund, road tunnel						
	7.1 Ellingsøy	1987	Precambrian gneiss	68m²	3.5km	1.1km	-140m
	Valderøy	1987	Precambrian gneiss	68m²	4.2km	2.2km	-137m

Though Stages 1 to 4 of the pre-investigations have been carried out for all these tunnels, core drilling has not been performed for three of them (Nos. 2, 7.1 and 7.2). The total cost of pre-investigations is between Nkr 2400-5400 per subsea metre of the tunnel (1987 price levels) (£218-490).

Predictions of rock quality are based on an overall interpretation of results from the various steps of preinvestigations. A major problem during this interpretation is the transformation of seismic data into geological descriptions. For several of the completed projects, such problems have caused considerable discrepancies between prediction and the actual conditions encountered during tunnelling.

One result from the research programme which illustrates this point is shown in Fig 3. Here, the relative lengths of major weakness zones which were estimated from the preinvestigations are plotted against the length of concrete lining in the actual tunnels. The comparison is based on the generally accepted assumption that concrete lining is necessary when a major weakness zone is crossed.

The majority of discrepancies in Fig 3 have geological explanations. For instance, for Tunnel 5.2 (Førdesfjorden) the total zone length was underestimated because of crushed zones running parallel to the tunnel axis. These zones were not identified due to the omission of seismic cross profiles. For Project 7.1 (Ellingsøyfjorden) the major reason for discrepancy was that a combination of systematic rock bolting and steel fibre reinforced shotcrete was used as an alternative to concrete lining.

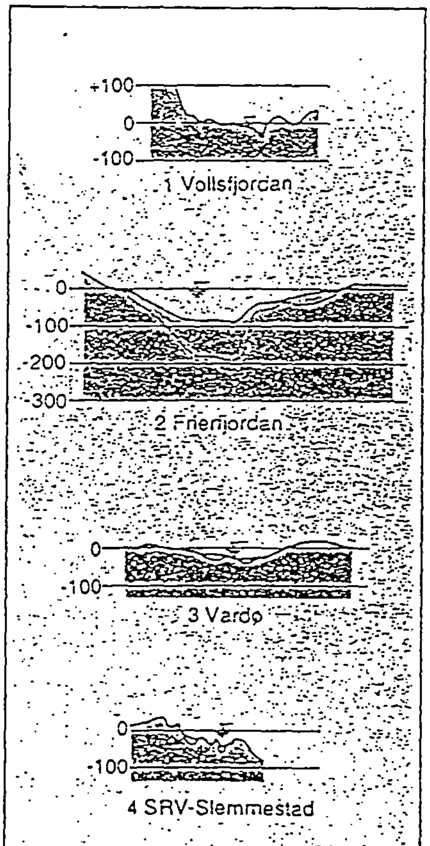
The estimated costs of rock support and grouting for these tunnels were exceeded by 20-110%. Although part of this discrepancy may have been caused by other factors, geological features are believed to be mainly responsible. A major conclusion from the state-of-the-art reviews is therefore that comprehensive geo-investigations are vital for all subsea projects, and the importance of thorough processing of all investigation result should not be underestimated.

Tunnelling

All projects to date have been tunnelled by using normal drill+blast techniques. The average tunnelled length/wk has varied between 17 and 40m/face (approximately 110 working hours/wk) (see Table 2).

As previously indicated, the rock conditions along the tunnel alignment will generate uncertainties even when comprehensive preinvestigations have been carried out. Continuous probe drilling is therefore necessary during subsea tunnelling to identify changing rock conditions and thereby increase the level of safety. This type of drilling may be regarded as a delayed part of the preinvestigations. Percussive probe drilling is done on a routine basis along the subsea part of the tunnel; typically the length drilled is 3-5 times the tunnel length. Some core drilling is also usual, but only to a lesser extent.

The major problems during tunnelling have occurred during the crossing of significant weakness zones. In many cases



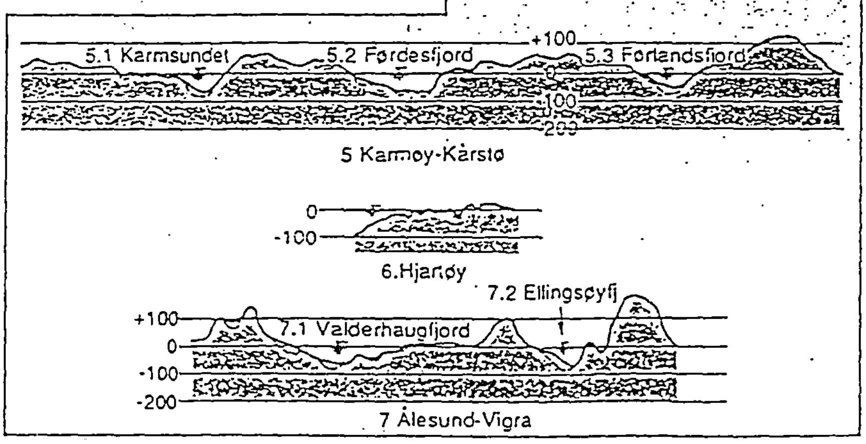


Fig 2. Longitudinal sections along the respective tunnels in Fig 1. Solid lines below water table indicate sea bed; dotted lines indicate rock surface.

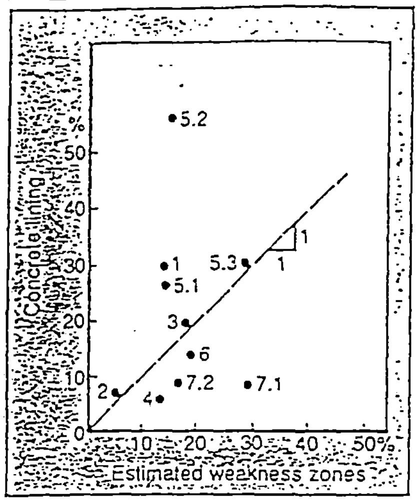


Fig 3. Relative lengths of estimated weakness zones under sea plotted against relative lengths of concrete-lined sections.

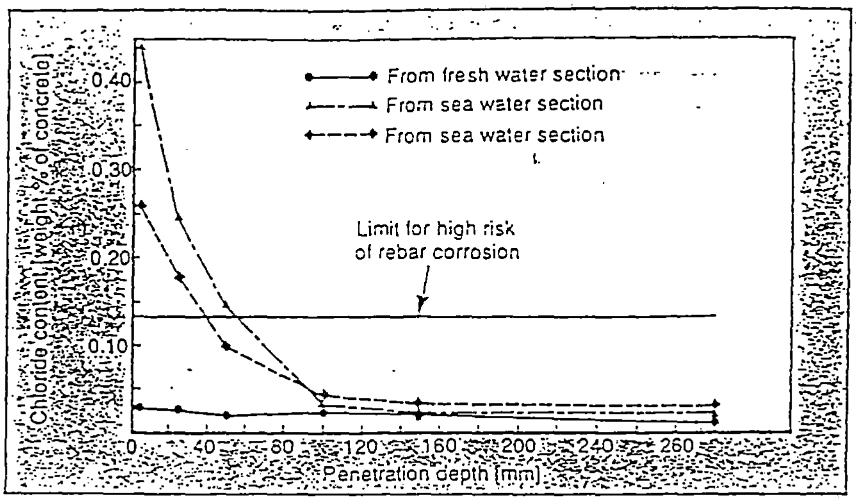


Fig 5. Chloride penetration profiles in cast concrete. Examples from the Vardø Tunnel.

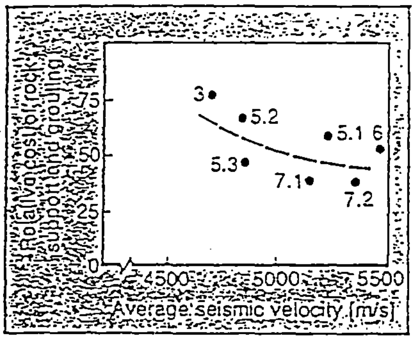


Fig 4. Relative costs of rock support and grouting as a function of average seismic velocity.

these zones contain very active swelling clay. Swelling pressures of up to 2.4MPa have been experienced. In most cases with major weakness zones concrete lining at the working face has been used as rock support.

Because of their high clay content, most weakness zones are practically impermeable. In some cases, however, even zones which are rich in clay may be waterbearing. Such zones represent the most difficult problems. Nevertheless, large, concentrated leakages have not been a major problem so far.

The grouting criterion which is most commonly used is based on measuring the volume of leakage water from the probe drill holes, and grouting is then required if the leakage exceeds a certain limit (normally 5-10 litre/min in a given borehole). The grout consumption per tunnelled metre varies from 2kg/m (Tunnel 5.3) to 98kg/m (Tunnel 7.1). Water leakages and grouting are discussed in more detail by Nilsen[6].

Primary and permanent shotcrete

Some key data concerning tunnelling rate, rock support and grouting are summarised in Table 2.

In Fig 4 the relative costs of rock support and grouting are plotted against the average seismic velocity recorded by refraction seismic profiling. As can be seen, the costs of rock support and grouting in many cases represent the major part of the total tunnelling cost. When the average seismic velocity decreases, there is a distinct tendency for the relative rock support and grouting costs to increase.

Rock support materials

The major types of rock support materials in the reported subsea tunnels are rock bolts, shotcrete (unreinforced and steel fibre reinforced), cast concrete, grout, insulated aluminium sheets and ethafoam (PE-foam). Some key data about the quantities of rock support are summarised in Table 2.

Grouted pipe bolts are most commonly used for rock bolting on the working face. Behind the working face, grouted rebar bolts are usual. All rock bolts are hot dip galvanised. The stipulated requirements for rock bolt types and work specifications have remained unaltered since 1975.

Shotcreting with and without steel fibre reinforcement has been used extensively and in most cases more than planned. Most of the shotcreting has been applied on the working face as combined working and permanent support, but considerable volumes have also been applied behind the working face as permanent support. There has been a considerable development in material quality. Due to higher load capacity and easier performance compared to mesh reinforced shotcreting, the use of steel fibre reinforcement has increased dramatically. The stipulated material requirement today for shotcrete in subsea tunnels is normally grade C 45 or a water-cement ratio lower than 0.45.

Cast in-situ concrete linings have been used mainly in poor quality rock situations on the working face. Concrete quality requirements have increased during the period in question, mainly due to environmental loads. For cast in-situ concrete linings, the stipulated requirement today is grade C45 or a water-cement ratio lower than 0.45.

Cement-based materials have been used mainly for grouting. Chemical grouts have been used only in a few cases. All cement grouting has been carried out on the working face. The stipulated requirements for grouting materials have not been increased since 1975.

Sheet vaulting has been used occasionally. In the Vardø Tunnel double aluminium sheets with insulation have

Table 2. Average tunnelling rates and some major data concerning rock support and grouting as recorded on the ten completed undersea tunnels in Norway.

Tunnel	Average tunnelled length (m)	Rock bolts (number/m)	Supported length as percentage of total length: Shotcrete (%)	Concrete lining (%)	Grouting (%)
1. Vollsfjord	26	>0.6 2)	6 2)	20	1)
2. Frierfjord	28	0.7 4)	4 2)	6	1)
3. Vardø	17	6.9	50	21	7
4. SRV	1)	1)	62	5	8
5.1. Karmsundet	34	1.5	65	15	9
5.2. Fordesfjord	26	2.3	35	33	6
5.3. Forlandsfjord	35	2.5	28	17	4
6. Hjartøy	40	1.6	33	12	10
7.1 Ellingsøy	28	6.4	20	3	22
7.2 Valderøy	33	5.0	58	5	6

1) No data available. 2) Uncertain data.

been used for water and frost protection. In the Ålesund tunnels PE-foam has been used as protection against water and frost.

Inspection of rock support

Due to the aggressive character of the leakage water in subsea tunnels, the environmental loads on support materials are very high. During the state-of-the-art review the in-situ condition of supporting materials was evaluated by studying the oldest accessible tunnels, i.e. the Vardø and Karmøy-Kårstø tunnels, completed in 1981 and 1983 respectively.

Visual inspections have revealed no notable corrosion of rock bolts. In the subsea section of the Vardø Tunnel, several test-bolts were placed freely in drillholes. After six years of exposure to seawater these bolts are in very good condition. Some permanent rock bolts have been tensioned up to yielding load without any resultant deformation in the bolts or the grout.

In the Vardø Tunnel, shotcreting of strength grade C 25 has deteriorated considerably due to penetration by seawater. The quality of this concrete was not documented and it was only used as a working support. The situation is much better in higher qualities (grade C 35), where no deterioration has been observed. However, on the exposed surface of the concrete, chemical analyses indicate an initial attack where seawater is seeping quite freely on the concrete surface. It is possible that this is only a surface problem since shotcrete normally has a low surface quality due to poor curing. No corrosion of steel fibres inside the shotcrete has been identified in spite of a relatively high chloride content.

Cast in-situ concrete is generally in a better condition than shotcrete when concretes of the same strength grade are compared. In sections with seeping seawater on the concrete surface, initial deterioration due to magnesium has been observed on the exposed surface of the concrete. In the same sections, the chloride content has been found to be higher on the exposed surface than the generally accepted limit for a high risk of rebar corrosion (Fig 5).

Grout materials based on cement may be attacked as a result of a low pH-level, sulphates and magnesium in seawater. The beginning of an attack may result in a tighter grout. However, if there is a process of deterioration, this may be observed by increased water inflow or by changes in the composition of the leakage water. Visual inspections have revealed no increased leakage over time. The chemical composition of leakage water is mainly identical with the composition of ordinary seawater. This indicates that the deterioration of cement-based grouting materials has been negligible.

The aluminium plates in the Vardø Tunnel do not show any general corrosion. At contact points between rails and popnails, however, some corrosion products can be observed, and where there is contact with concrete there is local corrosion due to the alkalinity of the concrete and the aggressiveness of seawater on aluminium. This problem may be avoided by preventing electrical contact between concrete and alumina plates.

Plans for further research

This state-of-the-art review provides a sound foundation for a further programme of research which is planned to include the following topics:

- A further evaluation of possibilities for improving and rationalising existing preinvestigation procedures.
- A comprehensive study of cases of roof-instability and major rockfalls.
- A review of criteria for optimising the rock cover.
- A closer study of the durability of rock bolts and grouts.
- The preparation of recommendations for rock support and shotcrete works.

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Acknowledgements

This paper is based on results from the SINTEF "Subsea tunnels" research programme, which was sponsored by major owners, contractors and consultants of Norwegian subsea tunnels. The participants in this project include: Norsk Hydro a.s., Statoil A/S, Norwegian Public Roads Administration, Norcem Cement A/S, Astrup Høyer A/S, Selmer Furuholmen A/S, Ing. A.B., Berdal A/S, Ing. Chr. F. Grøner A/S and Noteby A/S. The authors take this opportunity to thank all participants for their positive cooperation during the project period, and for granting permission to submit this paper.

OBITUARY

Leopold Müller, Salzburg, Austria

Jan 9, 1908 — Aug 1, 1988

Univ.-Prof. Baurat h.c. Dipl.-Ing. Dr. techn. Dr. mont. h.c. Honorary Citizen of Salzburg

Holder of The Ring of Salzburg County, Holder of The Golden Ring of Salzburg, Holder of The Golden Plate of Salzburg County, Holder of the Salzburg Science Award, Honorary Member of the Austrian Academy of Science, Corresponding member of the Bologna Academy of Science, Past-President of the International Society of Rock Mechanics, Honorary President of the Austrian Society of Geomechanics, Honorary Member of the Austrian Society of Engineering and Architecture, Honorary lecturer of the Universities of Salzburg and Munich, Associate lecturer of the University of Lahore, Holder of the Rock Mechanics Award, Holder of the Carl-Friedrich-Gauß-Medal, the Hans-Cloos-Medal, the Haidinger-Medal and the Ritter-von-Prechtl-Medal.

His international recognition did not come by chance. A very humane engineer and scientist, he not only founded geomechanics as the missing link between geology and civil engineering, he also brought a paralysed society of engineers of all disciplines to think and work together.

He often invited us, his pupils, and friends and colleagues from all technical and academic disciplines round his table, guiding those lost in mathematical ravines and mountains of steel and concrete back to the fruitful ways of overall thinking. The stimulating and sometimes very open discussions in his Salzburg Geomechanics Colloquy, which was founded in his own living-room 37 years ago, are still proving his philosophical approach.

Not all of us accepted or appreciated his point of view and this did, on occasion, give rise to bitter remarks, as on the last afternoon of the Geomechanics Colloquy in October 1987 when he said: It is nonsense to specify a 40 to 50cm thickness of shotcrete for shallow tunnels based purely on computer calculation, and it is even more absurd to build it!

He will be forever remembered as a sharp thinker, always encouraging, a warm friend, colleague and teacher. He crowned the numerous awards and tributes given on the occasion of his 80th birthday with a three-week trip to China.

This year, the 37th Geomechanics Colloquy, named the Leopold Müller Colloquy in his honour, will take place without him.

G Sauer

ORIGINAL

ACI STRUCTURAL JOURNAL COMMITTEE REPORT

Title no. 85-S52 ACI 544.4R

Design Considerations for Steel Fiber Reinforced Concrete

reported by ACI Committee 544

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The present state of development of design practices for fiber reinforced concrete and mortar using steel fibers is reviewed. Mechanical properties are discussed, design methods are presented, and typical applications are listed.

Keywords: beams (supports;) cavitation; compressive strength; concrete slabs; creep properties; fatigue (materials); fiber reinforced concretes; fibers; flexural strength; freeze-thaw durability; metal fibers; mortars (material); structural design.

CONTENTS

CHAPTER 1—INTRODUCTION

Steel fiber reinforced concrete (SFRC) and mortar made with hydraulic cements and containing fine or fine and coarse aggregates along with discontinuous discrete steel fibers are considered in this report. These materials are routinely used in only a few types of ap-

*Members of the subcommittee that prepared the report.
†Co-chairmen of the subcommittee that prepared the report.
‡Deceased.

Pertinent discussion will be published in the May-June 1989 *ACI Structural Journal* if received by Dec. 1, 1988.

plications at present (1988), but ACI Committee 544 believes that many other applications will be developed once engineers become aware of the beneficial properties of the material and have access to appropriate design procedures. The contents of this report reflect the experience of the committee with design procedures now in use.

The concrete used in the mixture is of a usual type, although the proportions should be varied to obtain good workability and take full advantage of the fibers. This may require limiting the aggregate size, optimizing the gradation, increasing the cement content, and perhaps adding fly ash or other admixtures to improve workability. The fibers may take many shapes. Their cross sections include circular, rectangular, half-round, and irregular or varying cross sections. They may be straight or bent, and come in various lengths. A convenient numerical parameter called the aspect ratio is used to describe the geometry. This ratio is the fiber length divided by the diameter. If the cross section is not round, then the diameter of a circular section with the same area is used.

The designer may best view fiber reinforced concrete as a concrete with increased strain capacity, impact resistance, energy absorption, and tensile strength. However, the increase in these properties will vary from substantial to nil depending on the quantity and type of fibers used; in addition, the properties will not increase at the same rate as fibers are added.

Several approaches to designing members with steel fiber reinforced concrete (SFRC) are available that are based on conventional design methods supplemented by special procedures for the fiber contribution. These methods generally modify the internal forces in the member to account for the additional tension from the fibers. When supported by full-scale test data, these approaches can provide satisfactory designs. The major differences in the proposed methods are in the determination of the magnitude of the tensile stress increase due to the fibers and in the manner in which the total force is calculated. Other approaches that have been used are often empirical, and they may apply only in certain cases where limited supporting test data have been obtained. They should be used with caution in new applications, only after adequate investigation.

Generally, for structural applications, steel fibers should be used in a role supplementary to reinforcing bars. Steel fibers can reliably inhibit cracking and improve resistance to material deterioration as a result of fatigue, impact, and shrinkage, or thermal stresses. A conservative but justifiable approach in structural members where flexural or tensile loads occur, such as in beams, columns, or elevated slabs (i.e., roofs, floors, or slabs not on grade), is that reinforcing bars must be used to support the total tensile load. This is because the variability of fiber distribution may be such that low fiber content in critical areas could lead to unacceptable reduction in strength.

In applications where the presence of continuous reinforcement is not essential to the safety and integrity of the structure, e.g., floors on grade, pavements, overlays, and shotcrete linings, the improvements in flexural strength, impact resistance, and fatigue performance associated with the fibers can be used to reduce section thickness, improve performance, or both.

ACI 318 does not provide for use of the additional tensile strength of the concrete in building design and, therefore, the design of reinforcement must follow the usual procedure. Other applications provide more freedom to take full advantage of the improved properties of SFRC.

There are some applications where steel fibers have been used without bars to carry flexural loads. These have been short-span elevated slabs, e.g., a parking garage at Heathrow Airport with slabs 3 ft-6 in. (1.07 m) square by 2½ in. (10 cm) thick, supported on four sides (Anonymous 1971). In such cases, the reliability of the members should be demonstrated by full-scale load tests, and the fabrication should employ rigid quality control.

Some full-scale tests have shown that steel fibers are effective in supplementing or replacing the stirrups in beams (Williamson 1978; Craig 1983; Sharma 1986). Although it is not an accepted practice at present, other full-scale tests have shown that steel fibers in combination with reinforcing bars can increase the moment capacity of reinforced concrete beams (Henager and Doherty 1976; Henager 1977a).

Steel fibers can also provide an adequate internal restraining mechanism when shrinkage-compensating cements are used, so that the concrete system will perform its crack control function even when restraint from conventional reinforcement is not provided. Fibers and shrinkage-compensating cements are not only compatible, but complement each other when used in combination (Paul et al. 1981). Guidance concerning shrinkage-compensating cement is available in ACI 223.1R.

ASTM A 820 covers steel fibers for use in fiber reinforced concrete. The design procedures discussed in this report are based on fibers meeting that specification.

Additional sources of information on design are available in a selected bibliography prepared by Hoff (1976-1982), in ACI publications SP-44 (1974) and SP-81 (1984), in proceedings of the 1985 U.S.-Sweden joint seminar edited by Shah and Skarendahl (1986), and the recent ACI publication SP-105 edited by Shah and Batson (1987).

For guidance regarding proportioning, mixing, placing, finishing, and testing for workability of steel fiber reinforced concrete, the designer should refer to ACI 544.3R.

CHAPTER 2—MECHANICAL PROPERTIES USED IN DESIGN

2.1—General

The mechanical properties of steel fiber reinforced concrete are influenced by the type of fiber; length-to-diameter ratio (aspect ratio); the amount of fiber; the strength of the matrix; the size, shape, and method of preparation of the specimen; and the size of the aggregate. For this reason, mixtures proposed for use in design should be tested, preferably in specimens representing the end use, to verify the property values assumed for design.

SFRC mixtures that can be mixed and placed with conventional equipment and procedures use from 0.5 to 1.5 volume percent* fibers. However, higher percentages of fibers (from 2 to 10 volume percent) have been used with special fiber addition techniques and placement procedures (Lankard 1984). Most properties given in this chapter are for the lower fiber percentage range. Some properties, however, are given for the higher fiber percentage mixtures for information in applications where the additional strength or toughness may justify the special techniques required.

Fibers influence the mechanical properties of concrete and mortar in all failure modes (Gopalaratnam and Shah 1987a), especially those that induce fatigue and tensile stress, e.g., direct tension, bending, impact, and shear. The strengthening mechanism of the fibers involves transfer of stress from the matrix to the fiber by interfacial shear, or by interlock between the fiber and matrix if the fiber surface is deformed. Stress is thus shared by the fiber and matrix in tension until the matrix cracks, and then the total stress is progressively transferred to the fibers.

Aside from the matrix itself, the most important variables governing the properties of steel fiber reinforced concrete are the fiber efficiency and the fiber content (percentage of fiber by volume or weight and total number of fibers). Fiber efficiency is controlled by the resistance of the fibers to pullout, which in turn depends on the bond strength at the fiber-matrix interface. For fibers with uniform section, pullout resistance increases with an increase in fiber length; the longer the fiber the greater its effect in improving the properties of the composite.

Also, since pullout resistance is proportional to interfacial surface area, nonround fiber cross sections and smaller diameter round fibers offer more pullout resistance per unit volume than larger diameter round fibers because they have more surface area per unit volume. Thus, the greater the interfacial surface area (or the smaller the diameter), the more effectively the fibers bond. Therefore, for a given fiber length, a high ratio of length to diameter (aspect ratio) is associated with high fiber efficiency. On this basis, it would appear that the fibers should have an aspect ratio high enough to insure that their tensile strength is approached as the composite fails.

Unfortunately, this is not practical. Many investigations have shown that use of fibers with an aspect ratio greater than 100 usually causes inadequate workability of the concrete mixture, non-uniform fiber distribution, or both if the conventional mixing techniques are used (Lankard 1972). Most mixtures used in practice

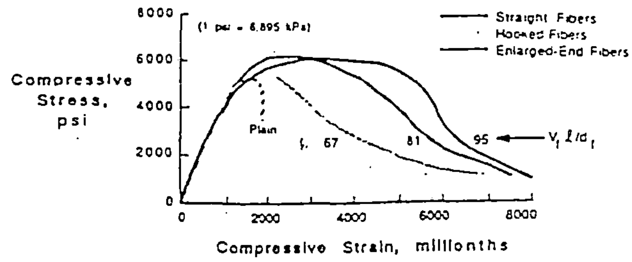


Fig. 2.1—Stress-strain curves for steel fiber reinforced concrete in compression, ⅜-in. (9.5-mm) aggregate mixtures (Shah 1978)

employ fibers with an aspect ratio less than 100, and failure of the composite, therefore, is due primarily to fiber pullout. However, increased resistance to pullout without increasing the aspect ratio is achieved in fibers with deformed surfaces or end anchorage; failure may involve fracture of some of the fibers, but it is still usually governed by pullout.

An advantage of the pullout type of failure is that it is gradual and ductile compared with the more rapid and possibly catastrophic failure that may occur if the fibers break in tension. Generally, the more ductile the steel fibers, the more ductile and gradual the failure of the concrete. Shah and Rangan (1970) have shown that the ductility provided by steel fibers in flexure was enhanced when the high-strength fibers were annealed (a heating process that softens the metal, making it less brittle).

An understanding of the mechanical properties of SFRC and their variation with fiber type and amount is an important aspect of successful design. These properties are discussed in the remaining sections of this chapter.

2.2—Compression

The effect of steel fibers on the compressive strength of concrete is variable. Documented increases for concrete (as opposed to mortar) range from negligible in most cases to 23 percent for concrete containing 2 percent by volume of fiber with $\ell/d = 100$, ¾-in. (19-mm) maximum-size aggregate, and tested with 6 x 12 in. (150 x 300 mm) cylinders (Williamson 1974). For mortar mixtures, the reported increase in compressive strength ranges from negligible (Williamson 1974) to slight (Fanella and Naaman 1985).

Typical stress-strain curves for steel fiber reinforced concrete in compression are shown in Fig. 2.1 (Shah et al. 1978). Curves for steel fiber reinforced mortar are shown in Fig. 2.2 and 2.3 (Fanella and Naaman 1985). In these curves, a substantial increase in the strain at the peak stress can be noted, and the slope of the descending portion is less steep than that of control specimens without fibers. This is indicative of substantially higher toughness, where toughness is a measure of ability to absorb energy during deformation, and it can be estimated from the area under the stress-strain curves or load-deformation curves. The improved toughness in compression imparted by fibers is useful in

* Percent by volume of the total concrete mixture.

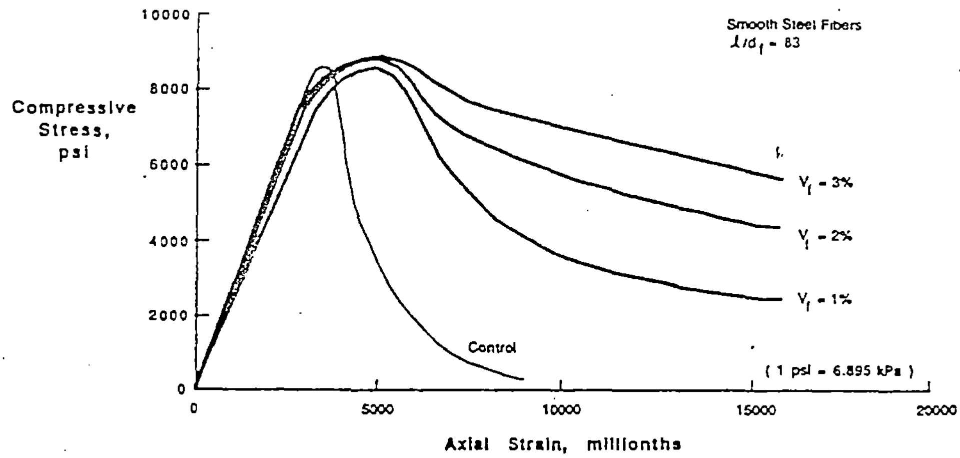


Fig. 2.2—Influence of the volume fraction of fibers on the compressive stress-strain curve

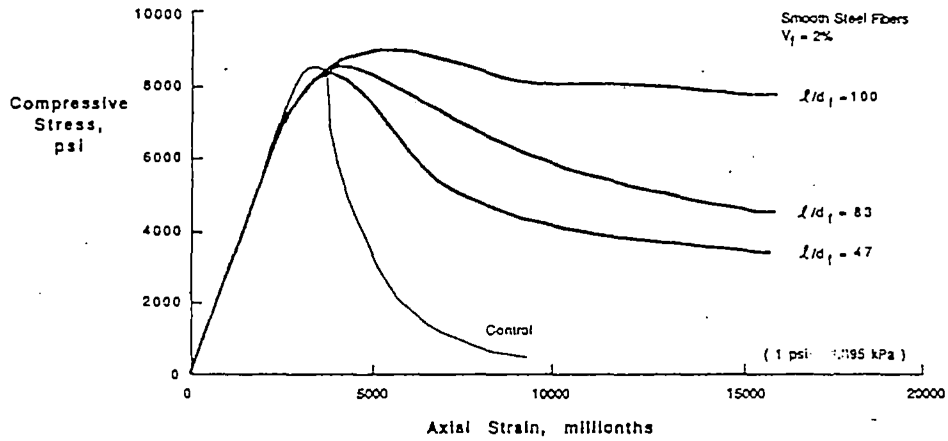


Fig. 2.3—Influence of the aspect ratio of fibers on the stress-strain curve

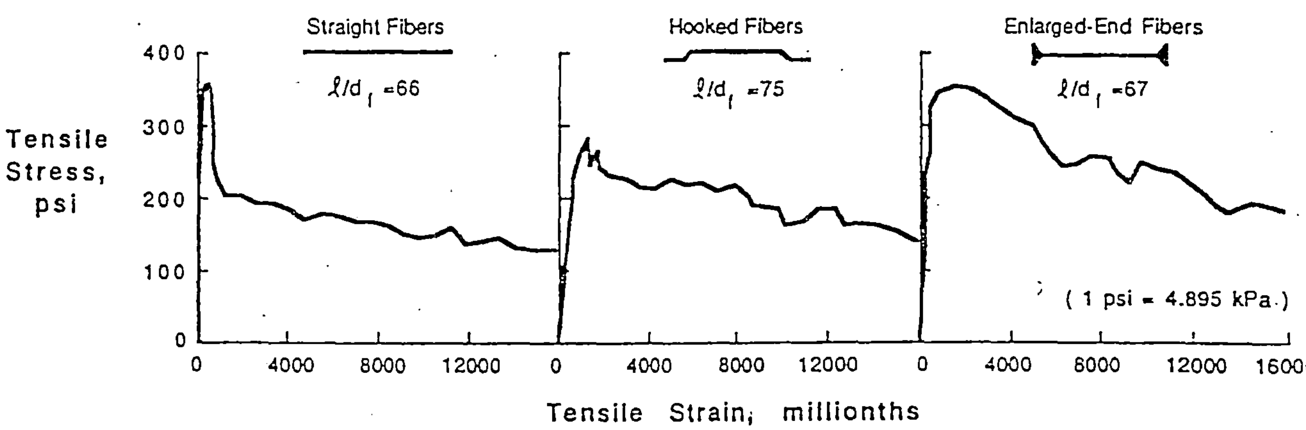


Fig. 2.4—Stress-strain curves for steel fiber reinforced mortars in tension (1.73 percent fibers by volume) (Shah 1978)

preventing sudden and explosive failure under static loading, and in absorbing energy under dynamic loading.

2.3—Direct tension

No standard test exists to determine the stress-strain curve of fiber reinforced concrete in direct tension. The observed curve depends on the size of the specimen, method of testing, stiffness of the testing machine, gage length, and whether single or multiple cracking occurs within the gage length used. Typical examples of stress-strain curves (with strains measured from strain gages) for steel fiber reinforced mortar are shown in Fig. 2.4 (Shah et al. 1978). The ascending part of the curve up to first cracking is similar to that of unreinforced mortar. The descending part depends on the fiber reinforcing parameters, notably fiber shape, fiber amount and aspect ratio.

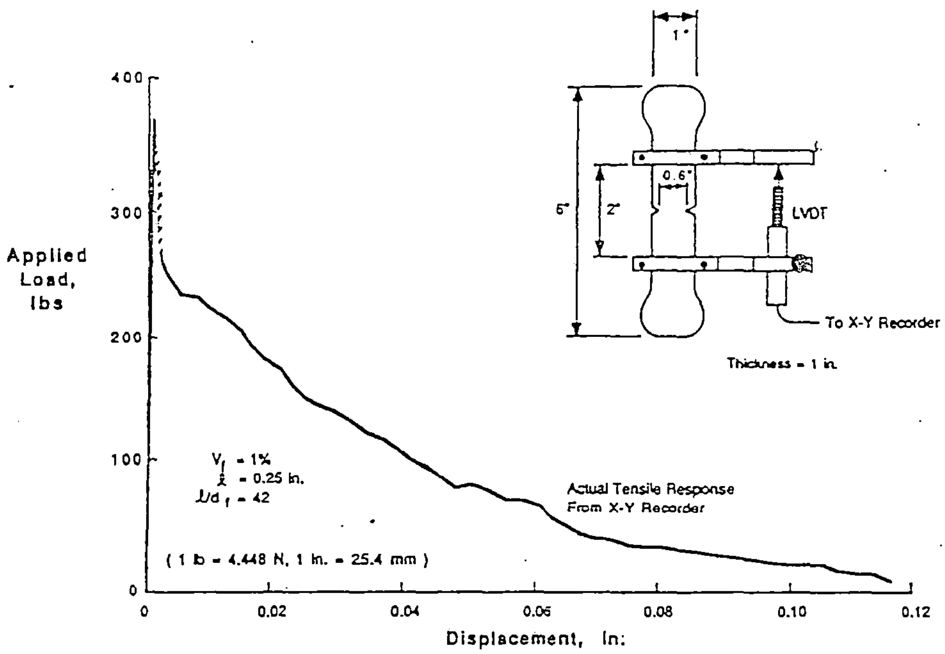


Fig. 2.5—Typical tensile load-versus-displacement curve of steel fiber reinforced mortar (Visalvanich and Naaman 1983)

An investigation of the descending, or post-cracking, portion of the stress-strain curve has led to the data shown in Fig. 2.5 and 2.6 and the prediction equation shown in Fig. 2.6 (Visalvanich and Naaman 1983). If only one crack forms in the tension specimen, as in the tests in Fig. 2.5, deformation is concentrated at the crack, and calculated strain depends on the gage length. Thus, post-crack strain information must be interpreted with care in the post-crack region (Gopalaratnam and Shah 1987b).

The strength of steel fiber reinforced concrete in direct tension is generally of the same order as that of unreinforced concrete, i.e., 300 to 600 psi (2 to 4 MPa). However, its toughness (as defined and measured according to ASTM C 1018) can be one to two orders of magnitude higher, primarily because of the large frictional and fiber bending energy developed during fiber pullout on either side of a crack, and because of deformation at multiple cracks when they occur (Shah et al. 1978; Visalvanich and Naaman 1983; Gopalaratnam and Shah 1987b).

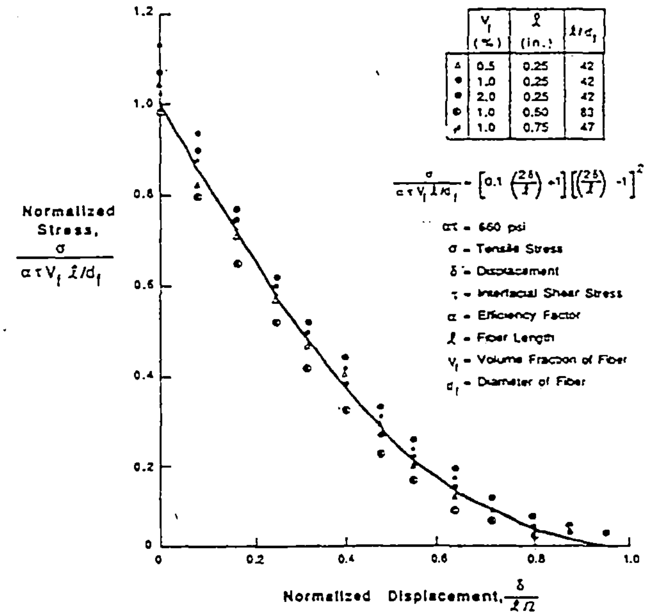


Fig. 2.6—Normalized stress-displacement law of steel fiber reinforced mortar (all cases) (Visalvanich and Naaman 1983)

2.4—Flexural strength

The influence of steel fibers on flexural strength of concrete and mortar is much greater than for direct tension and compression. Two flexural strength values are commonly reported. One, termed the first-crack flexural strength, corresponds to the load at which the load-deformation curve departs from linearity (Point A on Fig. 2.7). The other corresponds to the maximum load achieved, commonly called the ultimate flexural strength or modulus of rupture (Point C on Fig. 2.7). Strengths are calculated from the corresponding load using the formula for modulus of rupture given in ASTM C 78, although the linear stress and strain distributions on which the formula is based no longer apply after the matrix has cracked.

Fig. 2.8 shows the range of flexural load-deflection curves that can result when different amounts and types of fibers are used in a similar matrix and emphasizes the confusion that can occur in reporting of first-crack and ultimate flexural strength. For larger amounts of fibers the two loads are quite distinct (upper curve), but for smaller fiber volumes the first-crack load may be the maximum load as well (lower curves). The shape of

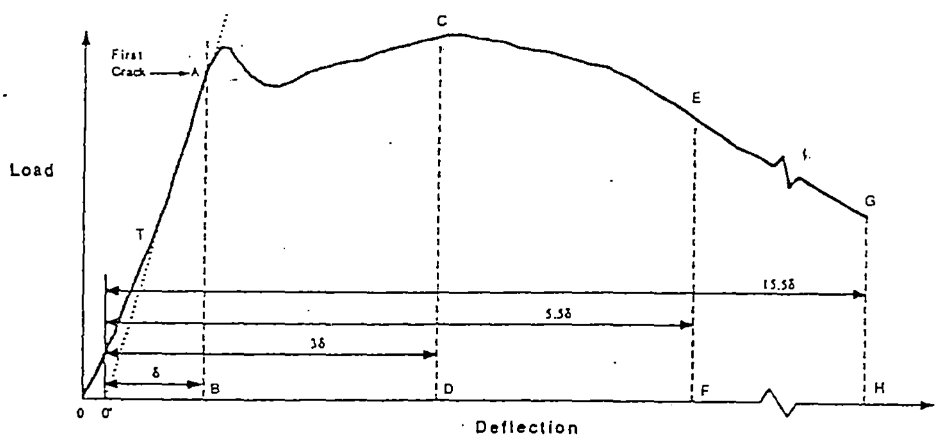
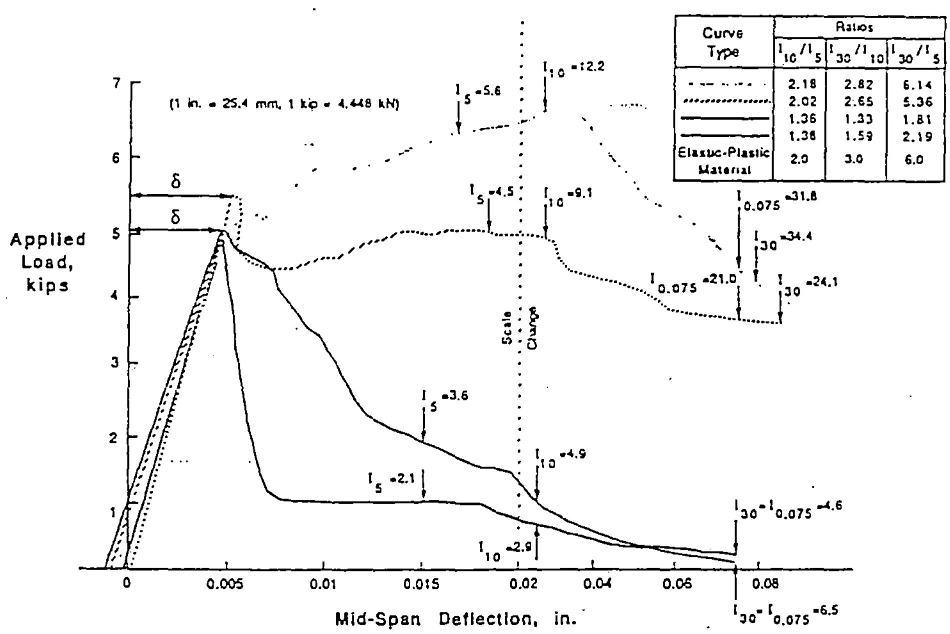


Fig. 2.7—Important characteristics of the load-deflection curve (ASTM C 1018)



Curve Type	Ratios		
	I_{10}/I_5	I_{30}/I_{10}	I_{30}/I_5
- - - - - -	2.18	2.82	6.14
···········	2.02	2.65	5.36
———	1.36	1.33	1.81
———	1.38	1.59	2.19
Elastic-Plastic Material	2.0	3.0	6.0

Fig. 2.8—Load-deflection curves illustrating the range of material behavior possible for four mixtures containing various amounts and types of fibers (Johnston 1982b)

the post-cracking curve is an important consideration in design, and this will be discussed relative to the calculation of flexural toughness. It is important, however, that the assumptions on which strength calculations are based be clearly indicated.

Procedures for determining first-crack and ultimate flexural strengths, as published in ACI 544.2R and ASTM C 1018, are based on testing 4 x 4 x 14 in. (100 x 100 x 350 mm) beams under third-point loading for quality control. Other sizes and shapes give higher or lower strengths, depending on span length, width and depth of cross section, and the ratio of fiber length to the minimum cross-sectional dimension of the test specimen.

It is possible, however, to correlate the results obtained in different testing configurations to values for standard beams tested under third-point loading, even when centerpoint loading is employed (Johnston 1982a). This is necessary when attempting to relate the performance of a particular design depth or thickness of material, e.g., a sample obtained from a pavement overlay or shotcrete lining, to the performance of standard 4 x 4 x 14 in. (100 x 100 x 350 mm) beams. The requirements relating cross-sectional size to design thickness of fiber reinforced concrete and to fiber length in ASTM C 1018 state that, for normal thickness of sections or mass concrete applications, the minimum cross-sectional dimension shall be at least three times the fiber length and the nominal maximum aggregate size.

Ultimate flexural strength generally increases in relation to the product of fiber volume concentration v and aspect ratio ℓ/d. Concentrations less than 0.5 volume percent of low aspect ratio fibers (say less than 50) have negligible effect on static strength properties. Prismatic fibers, or hooked or enlarged end (better anchorage) fibers, have produced flexural strength increases over unreinforced matrices of as much as 100 percent

(Johnston 1980). Post-cracking load-deformation characteristics depend greatly on the choice of fiber type and the volume percentage of the specific fiber type used. The cost effectiveness of a particular fiber type/amount combination should therefore be evaluated by analysis or prototype testing.

High flexural strengths are most easily achieved in mortars. Typical values for mortars (w/c ratio = 0.45 to 0.55) are in the range of 1000 to 1500 psi (6.5 to 10 MPa) for 1.5 percent by volume of fibers depending on the ℓ/d and the type of fiber, and may approach 1900 psi (13 MPa) for 2.5 percent by volume of fibers (Johnston 1980).

For fiber reinforced concretes, strengths decrease with increases in the maximum size and proportion of coarse aggregate present. In the field, workability considerations associated with conventional placement equipment and practices usually limit the product of fiber concentration by volume percent and fiber aspect ratio $v\ell/d$ to about 100 for uniform straight fibers. Twenty-eight day ultimate flexural strengths for concretes containing 0.5 to 1.5 percent by volume of fibers with ¼ to ¾ in. (8 to 19 mm) aggregate are typically in the range of 800 to 1100 psi (5.5 to 7.5 MPa) depending on $v\ell/d$, fiber type, and water-cement ratio.

Crimped fibers, surface-deformed fibers, and fibers with end anchorage produce strengths above those for smooth fibers of the same volume concentration, or allow similar strengths to be achieved with lower fiber concentrations. The use of a superplasticizing admixture may increase strengths over the value obtained without the admixture if the w/c ratio is reduced (Ramakrishnan and Coyle 1983).

2.5—Flexural toughness

Toughness is an important characteristic for which steel fiber reinforced concrete is noted. Under static loading, flexural toughness may be defined as the area under the load-deflection curve in flexure, which is the total energy absorbed prior to complete separation of the specimen (ACI 544.1R). Typical load-deflection curves for concrete with different types and amounts of fiber are shown in Fig. 2.8 (Johnston 1982b). Flexural toughness indexes may be calculated as the ratio of the area under the load-deflection curve for the steel fiber concrete to a specified endpoint, to the area up to first crack, as shown in ASTM C 1018, or to the area obtained for the matrix without fibers.

Some examples of index values computed using a fixed deflection of 0.075 in. (1.9 mm) to define the test endpoint for a 4 x 4 x 14 in. (100 x 100 x 350 mm) beam are shown in Fig. 2.8. Examples of index values I_5, I_{10}, and I_{30}, which can be computed for any size or shape of specimen, are also shown in Fig. 2.8.

These indexes, defined in ASTM C 1018, are obtained by dividing the area under the load-deflection curve, determined at a deflection that is a multiple of the first-crack deflection, by the area under the curve up to the first crack. I_5 is determined at a deflection 3 times the first-crack deflection, I_{10} is determined at 5.5, and I_{30} at 15.5 times the first-crack deflection. For example, for the second highest curve of Fig. 2.8, the first-crack deflection is 0.0055 in. (0.014 mm). I_5 is therefore determined at a deflection of 0.0165 in. (0.042 mm). The other values are computed similarly. ASTM C 1018 recommends that the end-point deflection and the corresponding index be selected to reflect the level of serviceability required in terms of cracking and deflection.

Values of the ASTM C 1018 toughness indexes depend primarily on the type, concentration, and aspect ratio of the fibers, and are essentially independent of whether the matrix is mortar or concrete (Johnston and Gray 1986). Thus, the indexes reflect the toughening effect of the fibers as distinct from any strengthening effect that may occur, such as an increase in first-crack strength.

Strengthening effects of this nature depend primarily on matrix characteristics such as water-cement ratio. In general, crimped fibers, surface-deformed fibers, and fibers with end anchorage produce toughness indexes greater than those for smooth straight fibers at the same volume concentration, or allow similar index values to be achieved with lower fiber concentrations. For concrete containing the types of fiber with improved anchorage such as surface deformations, hooked ends, enlarged ends, or full-length crimping, index values of 5.0 for I_5 and 10.0 for I_{10} are readily achieved at fiber volumes of 1 percent or less. Such index values indicate a composite with plastic behavior after first crack that approximates the behavior of mild steel after reaching its yield point (two upper curves in Fig. 2.8). Lower fiber volumes or less effectively anchored fibers produce correspondingly lower index values (two lower curves in Fig. 2.8).

2.6—Shrinkage and creep

Tests have shown that steel fibers have little effect on free shrinkage of SFRC (Hannant 1978). However, when shrinkage is restrained, tests using ring-type concrete specimens cast around a restraining steel ring have shown that steel fibers can substantially reduce the amount of cracking and the mean crack width (Malmberg and Skarendahl 1978; Swamy and Stavrides 1979). However, compression-creep tests carried out over a loading period of 12 months showed that the addition of steel fibers does not significantly reduce the creep strains of the composite (Edgington 1973). This behavior for shrinkage and creep is consistent with the low volume concentration of fiber when compared with an aggregate volume of approximately 70 percent.

2.7—Freeze-thaw resistance

Steel fibers do not significantly affect the freeze-thaw resistance of concrete, although they may reduce the severity of visible cracking and spalling as a result of freezing in concretes with an inadequate air-void system (Aufmuth et al. 1974). A proper air-void system (ACI 201.2R) remains the most important criterion needed to insure satisfactory freeze-thaw resistance, just as with plain concrete.

2.8—Abrasion/cavitation/erosion resistance

Both laboratory tests and full-scale field trials have shown that SFRC has high resistance to cavitation forces resulting from high-velocity water flow and the damage caused by the impact of large waterborne debris at high velocity (Schrader and Munch 1976a; Houghton et al. 1978; ICOLD 1982). Even greater cavitation resistance is reported for steel fiber concrete impregnated with a polymer (Houghton et al. 1978).

It is important to note the difference between erosion caused by impact forces (such as from cavitation or from rocks and debris impacting at high velocity) and the type of erosion that occurs from the wearing action of low velocity particles. Tests at the Waterways Experiment Station indicate that steel fiber additions do not improve the abrasion/erosion resistance of concrete caused by small particles at low water velocities. This is because adjustments in the mixture proportions to accommodate the fiber requirements reduce coarse aggregate content and increase paste content (Liu 1981).

2.9—Performance under dynamic loading

The dynamic strength of concrete reinforced with various types of fibers and subjected to explosive charges; dropped weights; and dynamic flexural, tensile, and compressive loads is 3 to 10 times greater than that for plain concrete (Williamson 1965; Robins and Calderwood 1978; Suaris and Shah 1984). The higher energy required to pull the fibers out of the matrix provides the impact strength and the resistance to spalling and fragmentation under rapid loading (Suaris and Shah 1981; Gokoz and Naaman 1981).

An impact test has been devised for fibrous concrete that uses a 10-lb (4.54-kg) hammer dropped onto a steel ball resting on the test specimen. The equipment used to compact asphalt concrete specimens according to ASTM D 1559 can readily be adapted for this test; this is described in ACI 544.2R. For fibrous concrete, the number of blows to failure is typically several hundred compared to 30 to 50 for plain concrete (Schrader 1981b).

Steel fiber reinforced beams have been subjected to impact loading in instrumented drop-weight and Charpy-type systems (Suaris and Shah 1983; Naaman and Gopalaratnam 1983; Gopalaratnam, Shah, and John 1984; Gopalaratnam and Shah 1986). It was observed that the total energy absorbed (measured from the load-deflection curves) by SFRC beams can be as much as 40 to 100 times that for unreinforced beams.

CHAPTER 3—DESIGN APPLICATIONS

3.1—Slabs

The greatest number of applications of steel fiber reinforced concrete (SFRC) has been in the area of slabs, bridge decks, airport pavements, parking areas, and cavitation/erosion environments. These applications have been summarized by Hoff (1976-1982), Schrader and Munch (1976b), Lankard (1975), Johnston (1982c), and Shah and Skarendahl (1986).

Wearing surfaces have been the most common application in bridge decks. Between 1972 and 1982, fifteen bridge deck surfaces were constructed with fiber contents from 0.75 to 1.5 volume percent. All surfaces but one were either fully or partially bonded to the existing deck, and most of these developed some cracks. In most cases, the cracks have remained tight and have not adversely affected the riding quality of the deck. A 3 in. (75 mm) thick unbonded overlay on a wooden deck was virtually crack-free after three years of traffic (ACI Committee 544, 1978). Periodic examination of the 15 projects has shown that the SFRC overlays have performed as designed in all but one case. Recently, latex-modified fiber reinforced concrete has been used successfully in seven bridge deck rehabilitation projects (Morgan 1983).

3.1.1 *Slabs on grade*—SFRC projects that are slabs on grade fall into two categories: overlays and new slabs on prepared base.

Many of the bonded or partially bonded experimental overlays placed to date without proper transverse control joints developed transverse cracks within 24 to 36 hours after placement. There are several causes for this. One is that there is greater drying shrinkage and heat release in the SFRC mixtures used because of the higher cement contents [of the order 800 lb/yd^3 (480 kg/m^3)] and the increased water demand. Recent designs have used much lower cement contents, thus reducing drying shrinkage.

It has been suggested that restrained shrinkage occurs in the overlay at a time when bond between the fiber and matrix is inadequate to prevent crack formation. In these cases, a suggested remedy is to use high-range water reducer technology and cooler placing temperatures. A study at the South Dakota School of Mines showed that drying shrinkage is reduced when the use of superplasticizers in SFRC results in a lower water-cement ratio. SFRC mixtures with w/c ratios less than 0.4 had lower shrinkage than conventional structural concrete mixtures (Ramakrishnan and Coyle 1983).

The most extensive and well monitored SFRC slab-on-grade project to date was an experimental highway overlay project in Green County, Iowa, constructed in September and October 1973 (Betterton and Knutson 1978). The project was 3.03 miles (4.85 km) long and included thirty-three 400 x 20 ft (122 x 6.1 m) sections of SFRC overlays 2 and 3 in. (50 and 75 mm) thick on badly broken pavement. Many major mixture and design variables were studied under the same loading and environmental conditions, and performance continues to be monitored.

Early observations on the Green County project indicated that the use of debonding techniques has greatly minimized the formation of transverse cracks. However, later examinations indicated that the bonded sections had outperformed the unbonded sections (Betterton and Knutson 1978). The 3 in. (75 mm) thick overlays are performing significantly better than those that are 2 in. (50 mm) thick. In the analysis of the Green County project, it was concluded that fiber content was the parameter that had the greatest impact on performance, with the higher fiber contents performing the best.

There are few well documented examples of the comparison of SFRC with plain concrete in highway slabs on grade. However, in those projects involving SFRC slabs subjected to heavy bus traffic, there is evidence that SFRC performed as well as plain concrete without fibers at SFRC thicknesses of 60 to 75 percent of the unreinforced slab thickness (Johnston 1984).

The loadings and design procedures for aircraft pavements and warehouse floors are different from those used for highway slabs. For nonhighway uses, the design methods for SFRC are essentially the same as those used for nonfiber concrete except that the improved flexural properties of SFRC are taken into account (AWI c. 1978; Schrader 1984; Rice 1977; Parker 1974; Marvin 1974; BDC 1975).

Twenty-three airport uses (Schrader and Lankard 1983) of SFRC and four experimental test slabs for aircraft-type loading have been reported. Most uses are overlays, although a few have been new slabs cast on prepared base. The airport overlays of SFRC have been constructed considerably thinner (usually by 20 to 60 percent) than a comparable plain concrete overlay would have been, and, in general, have performed well, as reported by Schrader and Lankard (1983) in a study on curling of SFRC. In those cases where comparison with a plain concrete installation was possible, as in the experimental sections, the SFRC performed significantly better.

The majority of the SFRC placements have shown varying amounts of curling at corners or edges (Schrader and Lankard 1983). The curling is similar to that evidenced by other concrete pavements of the same thickness reinforced with bar or mesh. Depending upon the amount of curling, a corner or edge crack may eventually form because of repeated bending. Thinner sections, less than 5 in. (125 mm), are more likely to exhibit curling.

The design of SFRC slabs on grade involves four considerations: (1) flexural stress and strength; (2) elastic deflections; (3) foundation stresses and strength; and (4) curl. The slab must be thick enough to accommodate the flexural stresses imposed by traffic and other loading. Since traffic-induced stresses are repetitive, a reasonable working stress must be established to insure performance under repeated loading.

In comparison with conventional concrete slabs, a fibrous concrete slab is relatively flexible due to its reduced thickness. The magnitude of anticipated elastic deflections must be assessed, because excessive elastic deflections increase the danger of pumping in the subgrade beneath the slab.

Stresses in the underlying layers are also increased due to the reduced thickness, and these must be kept low enough to prevent introduction of permanent deformation in the supporting materials.

Specific recommendations to minimize curl are available (Schrader and Lankard 1983). They include reducing the cement content, water content, and temperature of the plastic concrete, and using Type II portland cement, water reducing admixtures, and set-retarding admixtures. Other recommendations cover curing and construction practices and joint patterns.

The required slab thickness is most often based on a limiting tensile stress in flexure, usually computed by the Westergaard analysis of a slab on an elastic foundation. Selection of an appropriate allowable stress for the design is difficult without laboratory testing, because the reduction factor to account for fatigue and variability of material properties may be different for each mixture, aggregate, water-cement ratio, fiber type, and fiber content.

Parker (1974) has developed pavement thickness design curves for SFRC similar to the design curves for conventional concrete. For general SFRC, the ultimate flexural strength (modulus of rupture) is of the order 1.5 times that of ordinary concrete. A working value of 80 percent of the modulus of rupture obtained from the laboratory SFRC specimen has been conservatively suggested as a design parameter for aircraft pavements (Parker 1974). A value of two-thirds the modulus of rupture has been suggested for highway slabs.

Typical material property values for SFRC that has been used for pavements and overlays are: flexural strength = 900 to 1100 psi (6.2 to 7.6 MPa), compressive strength = 6000 psi (41 MPa), Poisson's ratio = 0.2, and modulus of elasticity = 4.0 x 10^6 psi (27,600 MPa). Typical mixtures that achieve properties in these ranges are shown in ACI 544.3R. Schrader (1984) has developed additional guidance for adapting existing pavement design charts for conventional concrete to the design of fiber reinforced concretes.

Flexural fatigue is an important parameter affecting the performance of pavements. The available data indicate that steel fibers increase the fatigue resistance of the concrete significantly. Batson et al. (1972b) found that a fatigue strength of 90 percent of the first-crack strength at 2 x 10^6 cycles to 50 percent at 10 x 10^6 cycles can be obtained with 2 to 3 percent fiber volume in mortar mixtures for nonreversal type loading. Morse and Williamson (1977), using 1.5 percent fiber volume, obtained 2 x 10^6 cycles at 65 percent of the first-crack stress without developing cracks, also for a nonreversal loading. Zollo (1975) found a dynamic stress ratio [ratio of first-crack stress that will permit 2 x 10^6 cycles to the static (one cycle) first-crack stress] for overlays on steel decks between 0.9 and 0.95 at 2 million cycles.

Generally, fatigue strengths are 65 to 95 percent at one to two million cycles of nonreversed load, as compared to typical values of 50 to 55 percent for beams without fibers. Fatigue strengths are lower for fully reversed loading. For properly proportioned high-quality SFRC, a fatigue value of 85 percent is often used in pavement design. The designer should use fatigue strengths that have been established for the fiber type, volume percent, approximate aggregate size, and approximate mortar content of the materials to be used. Mortar mixtures can accept higher fiber contents and do not necessarily behave the same as concrete mixtures.

3.1.2 *Structural floor slabs*—For small slabs of steel fiber reinforced concrete, Ghalib (1980) presents a design method based on yield line theory. This procedure was confirmed and developed from tests on one-way slabs ¾ in. thick by 6 in. wide by 20 in. long (19 x 150 x 508 mm) on an 18-in. (457-mm) span line loaded near the third points, and on two-way slabs 1.3 in. x 37.8 in. square (33 x 960 mm square) on a 35.4-in. (900-mm) span point loaded at the center. The design method applies to slabs of that approximate size only, and the designer is cautioned not to attempt extrapolation to larger slabs. Design examples given by Ghalib (1980) are for slabs about 0.78 in. (20 mm) thick.

3.1.3 *Bridge decks*—Deterioration of concrete bridge decks due to cracking, scaling, and spalling is a critical maintenance problem for the nation's highway system. One of the main causes of this deterioration is the intrusion of deicing salts into the concrete, causing rapid corrosion of the reinforcing. As discussed in Section 3.1, SFRC overlays have been used on a number of projects in an attempt to find a practical and effective method of prevention and repair of bridge deck deterioration. The ability of steel fibers to control the frequency and severity of cracking, and the high flexural and fatigue strength obtainable with SFRC can provide significant benefit to this application.

However, the SFRC does not stop all cracks, nor does it decrease the permeability of the concrete. As a consequence, SFRC by itself does not solve the problem of intrusion of deicing salts, although it may help by limiting the size and number of cracks. The corrosion of fibers is not a problem in sound concrete. They will corrode in the presence of chlorides, but their small size precludes their being a cause of spalling (Morse and Williamson 1977; Schupack 1985). See ACI 544.1R for additional data on steel fiber corrosion.

3.2—Flexure in beams

3.2.1 *Static flexural strength prediction for beams with fibers only*—Several methods have been developed to predict the flexural strength of small beams reinforced only with steel fibers (Schrader and Lankard 1983; Lankard 1972; Swamy et al. 1974). Some use empirical data from laboratory experiments. Others use the fiber bond area or the law of mixtures, plus a random distribution factor, bond stress, and fiber stress.

Equations developed by Swamy et al. (1974) have a form based on theoretical derivation with the coefficients obtained from a regression analysis of that data. Although the coefficient of correlation for the regression analysis (of the laboratory data analyzed) was 0.98, the predictions may be as much as 50 percent high for field-produced mixtures.

Concrete and mortar, a wide range of mixture proportions, fiber geometries, curing methods, and cement of two types were represented in data from several authors. The first coefficient in each equation should theoretically be 1.0. The equations are applicable only to small [4 x 4 x 12 in. (100 x 100 x 305 mm)] beams, such as those used in laboratory testing or as small minor secondary members in a structure. The designer should not attempt extrapolation to larger beams or to fiber volumes outside the normal range of the data used in the regression analysis. The equations are
first-crack composite strength, psi

$$\sigma_{cf} = 0.843 f_r V_m + 425 V_f \ell/d_f \qquad (3\text{-}1)$$

ultimate composite flexural strength, psi

$$\sigma_{cu} = 0.97 f_r V_m + 494 V_f \ell/d_f \qquad (3\text{-}2)$$

where

f_r = stress in the matrix (modulus of rupture of the plain mortar or concrete), psi
V_m = volume fraction of the matrix = $1 - V_f$
V_f = volume fraction of the fibers = $1 - V_m$
ℓ/d_f = ratio of the length to diameter of the fibers (aspect ratio)

These equations correlate well with laboratory work. However, as previously noted, if they are used to predict strengths of field placements, the predictions will generally be higher than the actual values by up to 50 percent.

3.2.2 *Static flexural analysis of beams containing bars and fibers*—A method has been developed (Henager and Doherty 1976) for predicting the strength of beams reinforced with both bars and fibers. This method is similar to the ACI ultimate strength design method. The tensile strength computed for the fibrous concrete is added to that contributed by the reinforcing bars to obtain the ultimate moment.

The basic design assumptions made by Henager and Doherty (1976) are shown in Fig. 3.1, and the equation for nominal moment M_n of a singly reinforced steel fibrous concrete beam is

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) + \sigma_t b (h - e) \left(\frac{h}{2} + \frac{e}{2} - \frac{a}{2}\right) \qquad (3\text{-}3)$$

$$e = [\epsilon_s \text{ (fibers)} + 0.003]\, c/0.003 \qquad (3\text{-}4)$$

where

$$\sigma_t = 1.12\, \ell/d_f \rho_f F_{be} \text{ (inch/pound units, psi) or} \qquad (3\text{-}5)$$

$$\sigma_t = 0.00772\, \ell/d_f \rho_f F_{be} \text{ (SI units, MPa)} \qquad (3\text{-}6)$$

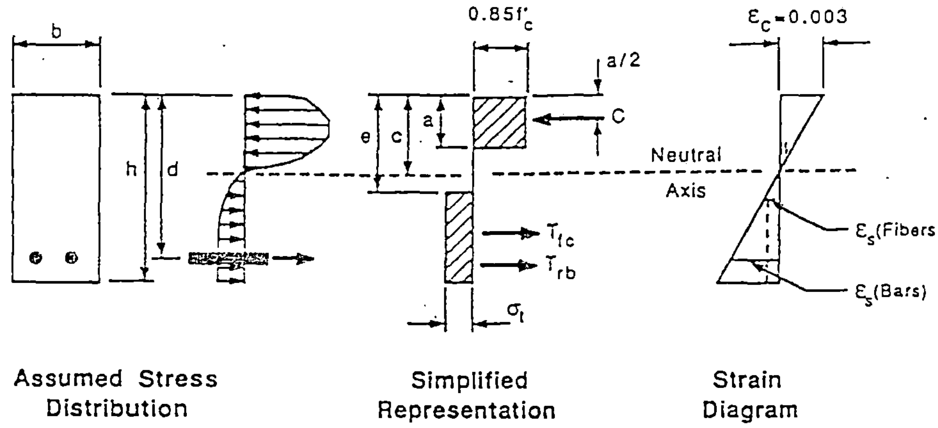


Fig. 3.1—Design assumptions for analysis of singly reinforced concrete beams containing steel fibers (Henager and Doherty 1976)

where

ℓ = fiber length
d_f = fiber diameter
ρ_f = percent by volume of steel fibers
F_{be} = bond efficiency of the fiber which varies from 1.0 to 1.2 depending upon fiber characteristics
a = depth of rectangular stress block
b = width of beam
c = distance from extreme compression fiber to neutral axis found by equating the internal tension and compression forces
d = distance from extreme compression fiber to centroid of tension reinforcement
e = distance from extreme compression fiber to top of tensile stress block of fibrous concrete (Fig. 3.1)
ϵ_s = tensile strain in steel at theoretical moment strength of beam, for bars = f_y/E_s; for fibers = σ_f/E_s based on fiber stress developed at pullout (dynamic bond stress of 333 psi) (Fig. 3.1)
ϵ_c = compressive strain in concrete
f'_c = compressive strength of concrete
f_y = yield strength of reinforcing bar
A_s = area of tension reinforcement
C = compressive force
h = total depth of beam
σ_t = tensile stress in fibrous concrete
E_s = modulus of elasticity of steel
T_{fc} = tensile force of fibrous concrete = $\sigma_t\, b\, (h - e)$
T_{rb} = tensile force of bar reinforcement = $A_s f_y$

In this analysis, the maximum usable strain at the extreme concrete compression fiber is taken to be 0.003. There are some data that indicate 0.003 may be conservative. Work by Williamson (1973) and Pearlman (1979) indicates that 0.0033 may be more realistic for steel fiber concrete. Swamy and Al-Ta'an (1981) recommend 0.0035. Based on a study of plastic hinges, Hassoun and Sahebjam (1985) recommend a failure strain of 0.0035 for concrete with 1.0 percent steel fibers, and 0.004 for 1 to 3 percent fibers.

The question arises as to whether the load factors and the capacity-reduction factor for flexure used in ACI 318 are still applicable. Normally, a smaller capacity-reduction factor would be used in the calculation of design strength when concrete tension is a major part of the resisting mechanism. In this use, however, the concrete tension contributes only about 5 to 15 percent of the resisting moment, which is significant but not a major part. Additional research is needed to define the reliability of the concrete tension force before a factor can be assigned to this type of member. It would be reasonable, however, to maintain a ϕ = 0.9 for the part of the resistance attributed to the deformed bar reinforcement [first term in Eq. (3-3)], and a smaller ϕ for the concrete tension contribution [second term in Eq. (3-3)].

The ratios of the calculated moments [using Eq. (3-3)] to actual moments in test beams ranged from 1.001 to 1.017 for a series of 6 beams reported by Henager and Doherty (1976). In these tests, a SFRC mortar mixture containing 940 lb of cement/yd^3 (557 kg/m^3), 2256 lb (1337 kg) of ¼-in. (6-mm) maximum size aggregate, and a w/c ratio of 0.45 or less was used. The method has also been applied successfully to fiber reinforced beams using a normal cement content [420 lb/yd^3 (250 kg/m^3)] and to beams of fiber reinforced lightweight aggregate concrete (Henager 1977a).

Eq. (3-5) and (3-6) incorporate a factor for bond stress of the fibers; this was chosen because it correlated with these tests. The selection of 333 psi (2.3 MPa) for bond stress was based on reported values in the range of 213 to 583 psi (1.5 to 4 MPa) for smooth, straight, round, high-strength fibers with embedment lengths of ½ to 1¼ in. (12 to 32 mm) (Williamson 1974; Aleszka and Beaumont 1973; Naaman and Shah 1976). This was combined with calculations that showed that 333 psi (2.3 MPa) would not cause fracture of the fibers used in the beams.

Fiber fracture rarely occurs in SFRC flexure loading with the fiber proportions and anchorage provisions normally available and with $\ell/d = 100$ or less. In this derivation the strain in the fibers is limited to the amount that produces about 333 psi, and it does not increase because the fibers slip and pull out. It is the pullout resistance that produces the toughness characteristic of SFRC during fracture. Other methods for static flexural analysis of beams containing bars and fibers have been proposed by Schrader (1971), Williamson (1973), Swamy and Al-Ta'an (1981), and Jindal (1984). There have been studies on combined axial load and flexure that deal with the same problem of including the effect of fibers on the tension force in the concrete (Craig et al. 1984b).

3.2.3 *Beam-to-column joints*—Additional studies related to flexure have been performed on beam-to-column connections. Henager (1977b) investigated the performance of a seismic-resistant beam-column joint using steel fibers in lieu of hoops in the joint region. Longitudinal steel bars were used in both the beam and the column. Deformed steel fibers 1½ x 0.020 in. (38 x 0.51 mm) were used at a fiber content of 1.67 percent by volume in the joint region, an area of high shear stresses.

In comparison to a conventional joint using hoop ties at 4 in. (100 mm) on centers, the SFRC joint showed no cracking in the joint region, whereas the conventional joint showed some hairline cracking. The SFRC joint developed a maximum moment of 56.5 kip-ft (76.7 kN-m) compared to 45.9 kip-ft (62.2 kN-m) for the conventional joint. The 28-day compressive strengths were 5640 psi (38.9 MPa) for the SFRC and 5915 psi (40.8 MPa) for the conventional concrete in the joint regions. Flexural strengths were 1419 psi (9.8 MPa) for the SFRC and 450 psi (3.1 MPa) for the conventional concrete.

Craig et al. (1984a) tested 10 joints, 5 of which contained steel fibers and a reduced quantity of deformed bar hoops. He also noted considerable improvement in the joint strength, ductility, and energy absorption with the steel fibers.

3.2.4 *Flexural fatigue considerations*—Batson et al. (1972b) recommended that 67 percent of the first-crack stress be used for 10^6 cycles of load in conventionally reinforced SFRC beams. Schrader (1971) has shown that the post-fatigue load-carrying capacity of SFRC beams is improved, but that the presence of conventional reinforcing bars overshadows the fatigue and static strength improvements obtained when comparing SFRC beams to beams with no conventional reinforcing.

Kormeling, Reinhardt, and Shah (1980) tested conventionally reinforced concrete beams with and without fibers in fatigue loading up to 10 million cycles. It was observed that the addition of fibers to conventionally reinforced concrete beams increased the fatigue life and decreased deflections and crack widths for a given number of dynamic cycles. The beneficial effect of fibers decreased with increasing volume of conventional reinforcement.

3.3—Shear in beams

There are considerable laboratory data indicating that fibers substantially increase the shear (diagonal tension) capacity of concrete and mortar beams. Steel fibers show several potential advantages when used to supplement or replace vertical stirrups or bent-up steel bars. These advantages are: (1) the fibers are randomly distributed through the volume of the concrete at much closer spacing than can be obtained with reinforcing bars; (2) the first-crack tensile strength and the ultimate tensile strength are increased by the fibers; and (3) the shear-friction strength is increased.

It is evident from a number of tests that stirrup and fiber reinforcement can be used effectively in combination. However, although the increase in shear capacity has been quantified in several investigations it has not yet been used in practical applications. This section presents the results of some of the studies dealing with the effect of steel fibers on shear strength in beams and slabs. It is important to identify the type and size of fiber upon which the design is based.

Batson et al. (1972a), using mortar beams 4 x 6 x 78 in. (100 x 150 x 2000 mm), conducted a series of tests to determine the effectiveness of straight steel fibers as web reinforcement in beams with conventional flexural reinforcement. In tests of 96 beams, the fiber size, type, and volume concentration were varied, along with the shear-span-to-depth ratio a/d, where a = shear span (distance between concentrated load and face of support) and d = the depth to centroid of reinforcing bars. (Shear capacity of rectangular beams may be considered a function of moment-to-shear ratio a/d or M/Vd.) Third-point loading was used throughout the test program.

It was found that, for a shear-span-to-depth ratio of 4.8, the nonfiber beams failed in shear and developed a shear stress at failure of 277 psi (1.91 MPa). For a fiber volume percent of 0.88, the average shearing stress at failure was 310 psi (2.14 MPa) with a moment-shear failure; for 1.76 volume percent, 330 psi (2.28 MPa) with a moment failure; and for 2.66 volume percent, 352 psi (2.43 MPa), also with a moment failure. The latter value represents an increase of 27 percent over the nonfiber beams. The shear stress at failure for beams with #3 [⅜-in. (9.5-mm) diameter] stirrups at 2-in. (50-mm) spacing in the outer thirds averaged 315 psi (2.17 MPa). All shearing stresses were computed by the equation $v = VQ/Ib$.

It was found that as the shear-span ratio decreased and fiber volume increased, higher shear stresses were developed at failure. For example, for an a/d of 3.6 and a volume percent of fiber of 0.88, the shear stress at failure was 444 psi (3.06 MPa) with a moment failure; for an a/d of 2.8 and a fiber volume percent of 1.76, the shear stress at failure was 550 psi (3.79 MPa) and a moment failure.

Paul and Sinnamon (1975) studied the effect of straight steel fibers on the shear capacity of concrete in a series of seven tests similar to those of Batson et al. (1972a). The objective was to determine a procedure for

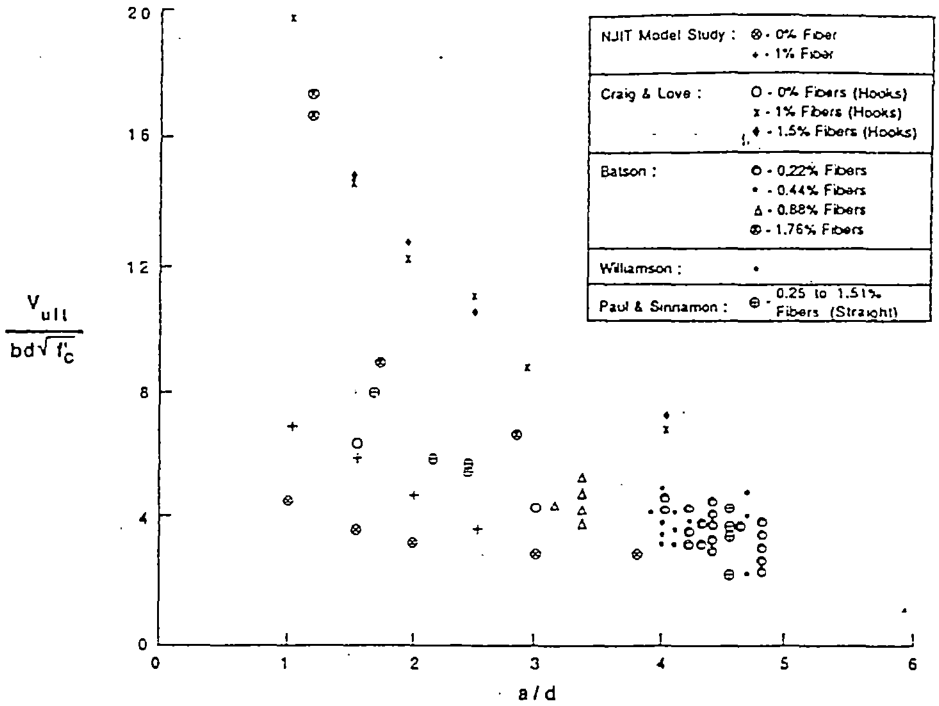


Fig. 3.2—Shear behavior of reinforced fibrous concrete beams

predicting the shear capacity of segmented concrete tunnel liners made with steel fit·‥r reinforced concrete. Their results agreed closely with Batson, especially for beams with similar a/d ratios.

Williamson (1978), working with conventionally, reinforced beams 12 x 21.5 in. x 23 ft (305 x 546 x 7010 mm), found that when 1.66 percent by volume of straight steel fibers were used in place of stirrups, the shear capacity of the beams was increased 45 percent over a beam without stirrups. Nevertheless, the beams failed in shear. This is consistent with the results of other investigators. When steel fibers with deformed ends were used (1.1 percent by volume), the shear capacity was increased by 45 to 67 percent and the beams failed in flexure.

Williamson (1978) concluded that, based upon the use of steel fibers with deformed ends, steel fibers can increase the shear strength of concrete beams enough to prevent catastrophic diagonal tension failure and to force the beam to fail in flexure. In his report, Williamson (1978) presents an analysis showing that steel fibers can present an economical alternative to the use of stirrups in reinforced concrete design.

Tests of crimped-end fibers have shown considerable increase in the shear capacity of reinforced concrete in other studies. Some of the tests at the New Jersey Institute of Technology (Craig 1983) have shown increases of more than 100 percent. Twelve full-scale test beams with 1.0 and 1.5 percent by volume of 0.020 x 1.18 in. (0.5 x 30 mm) long crimped-end fibers were tested with the following span-to-depth ratios: a/d = 1.0, 1.5, 2.0, 2.5, and 3.0. The beams had a 6 x 12 in. (150 x 300 mm) section. The increases in shear capacity for the 1.0 and 1.5 percent fiber content with a/d = 1.5 were 1: and 140 percent, respectively. Similarly, the increase : a/d = 3.0 was 108 percent for 1.0 volume percent ‹ fiber. The combination of stirrups and fibers showe slow and controlled cracking and better distribution ᶜ tensile cracks, and minimized the penetration of sheꜹ cracks into the compression zone.

It was also found that when fibers with crimped end were the only shear reinforcement, there was a signif cant decrease in diagonal tension cracking in the beam: Fig. 3.2 shows the results of the tests reported by Crai (1983) and compares them with other test results.

Bollano (1980) investigated the behavior of steel f bers as shear reinforcement in two-span continuousl reinforced concrete beams. These tests indicate the be havior in shear for the common range of M/Vd ratic for negative moment regions (M/Vd = 2 to 3, equiva lent to a/d for simple beams). It is generally assume that the M/Vd concept can be used equally well in sim ply supported and continuous beams, but this is nc entirely true for the beams investigated. The a/d rati was 4.8 and the M/Vd ratio was 3.0. The regular rein forced concrete beam $V/bd\sqrt{f'_c}$ ranged from 3 to 4 whereas this parameter for the beams with straight an crimped-end fibers ranged from 5 to 8, showing signil icant improvement with the addition of fibers.

Criswell (1976) conducted a number of differen shear tests, all of which demonstrated an increase shear capacity with the use of steel fibers. All of hi tests were made with concrete containing 1.0 percent b volume of straight fibers. The results of four shear friction specimens showed a 20 percent increase in shea strength; bolt pullout tests showed a shear strength i:

excess of 64 percent greater than that for the nonfiber concrete; slab-column connection specimens developed shearing strengths 27 percent greater than the nonfiber specimens; and beam-column shear tests resulted in shear strengths up to 60 percent greater.

Sharma (1986) tested 7 beams with steel fiber reinforcement, of which 4 also contained stirrups. The fibers had deformed ends. Based on these tests and those by Batson et al (1972a) and Williamson and Knab (1975), he proposed the following equation for predicting the average shear stress v_{cf} in the SFRC beams. (In the equation that follows, a typographical error in Sharma's 1986 paper has been corrected.)

$$v_{cf} = \frac{2}{3} f_t' \left(\frac{d}{a}\right)^{0.25} \qquad (3\text{-}7)$$

where f_t' is the tensile strength of concrete obtained from results of indirect tension tests of 6 x 12 in. (150 x 300 mm) cylinders, and d/a is the effective depth-to-shear-span ratio. Straight, crimped, and deformed-end fibers were included in the analysis and the average ratio of experimental to calculated shear stress was 1.03 with a mean deviation of 7.6 percent. The influence of different fiber types and quantities is considered through their influence on the parameter f_t'. The proposed design approach follows the method of ACI 318 for calculating the contribution of stirrups to the shear capacity, to which is added the resisting force of the concrete calculated from the shear stress given by Eq. (3-7).

An additional design procedure for shear and torsion in composite reinforced concrete beams with fibers has been published by Craig (1986).

3.4—Shear in slabs

The influence of steel fiber reinforcement on the shear strength of reinforced concrete flat plates was investigated by Swamy et al. (1979) in a test series on four slabs with various fiber contents (0, 0.6, 0.9, and 1.2 percent by volume). The slabs were 72 x 72 x 5 in. (1830 x 1830 x 125 mm) with load applied through a square column stub 6 x 6 x 10 in. (150 x 150 x 250 mm). All slabs had identical tension and compression reinforcement, and the steel fibers had crimped ends and were 0.02 x 2 in. (0.5 x 50 mm) long. The shear strength increases were 22, 35, and 42 percent for the 0.6, 0.9, and 1.2 percent by volume fiber contents, respectively.

3.5—Shotcrete

Steel fiber shotcrete has been used in the construction of dome-shaped structures using the inflation/foam/shotcrete process (Williamson et al. 1977; Nelson and Henager 1981). Design of the structures follows the conventional structural design procedures for concrete domes, taking into account the increased compressive, shear, and flexural properties of fibrous concrete.

This material is also used for underground support and linings, rock slope stabilization, repair of deteriorated concrete, etc. (Kobler 1966; Shah and Skarendahl 1986; Morgan and McAskill 1984). A research effort carried out in a side chamber of an Atlanta subway station to examine shotcrete support in loosening rock is reported by Fernandez-Delgado et al. (1981).

A significant quantity of steel fiber reinforced shotcrete has been used throughout the world, and a state-of-the-art report has been prepared by ACI Committee 506 (ACI 506.1R). That report also contains information on material properties, application procedures, and mixtures.

3.6—Cavitation erosion

Failure of hydraulic concrete structures is often precipitated by cavitation-erosion failure of the concrete. SFRC was used to repair severe cavitation-erosion damage that occurred in good quality conventional concrete after relatively short service at Dworshak, Libby, and Tarbella Dams (ICOLD 1982; Schrader and Munch 1976a). All three are high-head structures capable of large flows and discharge velocities in excess of 100 fps (30.5 mps).

At Libby and Dworshak, both the outlet conduits and stilling basins were repaired. At Tarbella, fiber concrete was used as topping in the basin and ogee curve leading from the outlet conduit to the basin. All three projects have performed well since the repairs. It should be noted, however, that while SFRC improves resistance to erosion from cavitation, it does not improve resistance to erosion from abrasion or scouring (see Section 2.8).

3.7—Additional applications

There are several applications of SFRC that have involved a considerable volume of material, but which do not have well defined design methods specifically for SFRC. Among these are fence posts, sidewalks, embankment protection, machinery foundations, machine tool frames, manhole covers, dolosse, bridge deck expansion joints (nosings at joints to improve wear and impact resistance), dams, electric power manholes, ditch linings, mine cribbing, liquid storage tanks, tilt-up wall construction, and thin precast members (see also Shah and Batson 1987).

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American Concrete Institute
P. O. Box 19150
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ASTM
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CHAPTER 5 — NOTATION

a = depth of rectangular stress block
a = shear span, distance between concentrated load and face of support
A_s = area of tension reinforcement bars
b = width of beam

b_w = web or width of a rectangular beam
c = distance from extreme compression fiber to neutral axis
C = compressive force
d = distance from extreme compression fiber to centroid of tension reinforcement
d_f = fiber diameter (for a noncircular fiber, an equivalent fiber diameter is the diameter of a circle with the same area as the fiber)
e = distance from extreme compression fiber to top of tensile stress block of fibrous concrete
E = modulus of elasticity
E_s = modulus of elasticity of steel
f'_c = compressive strength of concrete
f_{ct} = splitting tensile strength
f_r = modulus of rupture
f_y = yield strength of reinforcing bar
F_{be} = bond efficiency factor
h = total depth of beam
I = moment of inertia of section
M_n = nominal moment capacity of section
M_u = factored moment at beam section
ℓ = fiber length
ℓ/d_f = aspect ratio = fiber length/fiber diameter
Q = first statical moment of an area about the neutral axis
T_{fc} = tensile force of fibrous concrete = $\sigma_t b (h - e)$
T_s = tensile force of bar reinforcement = $A_s f_y$
ν = fiber volume concentration or volume fraction (not percentage)
v = shear stress at section
v_{cf} = average shear stress in SFRC beam
V = shear force at section
V_c = nominal shear strength provided by concrete
V_f = volume fraction of fibers $(1 - V_m)$
V_m = volume fraction of the matrix $(1 - V_f)$
V_u = factored shear force at beam section
ϵ_c = compressive strain in concrete
ϵ_s = tensile strain in steel
σ_{cf} = first crack composite flexural strength
σ_{cu} = ultimate composite flexural strength
σ_f = tensile stress in fiber
σ_t = tensile stress in fibrous concrete
τ_d = dynamic bond stress between fiber and matrix
ρ_f = percent by volume of fibers
ρ_w = $A_s/b_w d$
ϕ = capacity reduction factor

This report was submitted to letter ballot of the committee and was approved in accordance with ACI balloting requirements.

improvements in many of the engineering properties of mortars and concrete. Impact strength is greatly improved as is the toughness. The flexural strength, fatigue strength, and the ability to resist cracking and spalling are also enhanced. More detailed information on properties may be found in references listed in Chapter 8.

1.3 — Typical uses of steel fiber reinforced concrete

Generally, when used in structural applications, steel fiber reinforced concrete should only be used in a supplementary role to inhibit cracking, to improve resistance to impact or dynamic loading, and to resist material disintegration. In structural members where flexural or tensile loads will occur, such as in beams, columns, suspended floors, (i.e., floors or slabs not on grade) the reinforcing steel must be capable of supporting the total tensile load. In applications where the presence of continuous reinforcement is not essential to the safety and integrity of the structure, e.g., pavements, overlays, and shotcrete linings, the improvements in flexural strength associated with the fibers can be used to reduce section thickness or improve performance, or both. The following are some examples of structural and nonstructural uses of SFRC:

- Hydraulic structures — Dams, spillways, stilling basins, and sluiceways as new or replacement slabs or overlays to resist cavitation damage. See Reference 5.
- Airport and highway paving and overlays — Particularly where a thinner than normal slab is desired.
- Industrial floors — For impact resistance and resistance to thermal shock.
- Refractory concrete — Using high-alumina cement in both castable and shotcreted applications. See References 6 and 7.
- Bridge decks — As an overlay or topping where the primary structural support is provided by an underlying reinforced concrete deck.
- In shotcrete linings — For underground support in tunnels and mines, usually with rock bolts.
- In shotcrete coverings — To stabilize aboveground rock or soil slopes, e.g., highway and railway cuts and embankments. See Reference 8.
- Thin shell structures — Shotcreted "foam domes."
- Explosion-resistant structures — Usually in combination with reinforcing bars.
- A possible future use is in seismic-resistant structures.

1.4 — Specifying steel fiber reinforced concrete

1.4.1 *General* — Steel fiber reinforced concrete is usually specified by strength and fiber content. The flexural strength is normally specified for paving applications while compressive strength is normally specified for structural applications. A flexural strength of 700 to 1000 psi (4.8 to 6.9 MPa) at 28 days and a compressive strength of 5000 to 7000 psi (34.5 to 48.3 MPa) are typical values. In general the addition of fibers does not significantly increase compressive strength but does increase the compressive strain at ultimate load. The changes in mixture proportions to accommodate the fibers do have a significant influence on the compressive strength.

For normal weight concrete, fiber contents have been used from as low as 50 lb/yd^3 (30 kg/m^3) to as high as 265 lb/yd^3 (157 kg/m^3) although the high range limit is usually about 160 to 200 lb/yd^3 (95 to 118 kg/m^3). Steel fibers have also been used in lightweight concrete, but data are not generally available. The amount that can be used, and the amount needed for a particular application, depends upon the fiber shape and aspect ratio* as well as the end use. See References 1 through 4 and 10 or consult fiber manufacturers for additional information. In terms of volume percentage for normal weight concrete, 50 lb/yd^3 = 0.38 percent by volume; 160 lb/yd^3 = 1.2 percent by volume; and 265 lb/yd^3 = 2.0 percent by volume.

Toughness, which is the area under a load-deflection curve, or a toughness index, which is a function of that area, may be determined to help define the performance requirements of SFRC intended for use where post-cracking energy absorption or resistance to failure after cracking is important. These properties would be important in applications such as structures subjected to earthquakes or explosive blasts, impact loads, cavitation loads, thermal shock, and other dynamic loads. A standard test for flexural toughness is being prepared by the ASTM subcommittee on Fiber Reinforced Concrete. After such a standard test has been adopted and more experience gained in the relationship of toughness to performance, it may be practical to use toughness as a performance standard for SFRC.

As noted in subsequent chapters, the manufacture and placing of SFRC is very similar to conventional concrete. Most existing concrete specifications can be used for the manufacture and placement of SFRC with some added requirements to account for the differences in materials and techniques. The subsequent chapters point out those differences.

1.4.2 *Guidelines for specifying ready-mixed SFRC using ASTM C 94* — ASTM C 94 may be used to specify steel fiber reinforced concrete. When using ASTM C 94, the purchaser should state that mixture proportions should be in accordance with Alternative No. 1, 2, or 3, except that flexural strength at 28 days may be specified for the strength requirement instead of compressive strength. For structures, however, the compressive strength may govern. See Section 1.4.1. Flexural strength should be determined in accordance with ASTM C 78 using 4 × 4 × 14 in. specimens on a 12-in. span (100 × 100 × 350 mm - 300 mm span) as recommended in ACI 544.2R. See ACI 544.2R for additional guidance on testing, e.g., specimens representing the design depth. The alternative methods in ASTM C 94 as applied to this guide are

*The aspect ratio of a fiber is its length divided by its diameter, l/d.

ORIGINAL

Guide For Specifying, Mixing, Placing, and Finishing Steel Fiber Reinforced Concrete

Reported by ACI Committee 544

This guide describes the current technology in specifying, mixing, placing, and finishing of steel fiber reinforced concrete (SFRC). Much of the current conventional concrete practice applies to SFRC. The emphasis in this guide is to describe the differences between conventional concrete and SFRC and how to deal with them. Guidance is provided in mixing techniques to achieve uniform mixtures, placement techniques to assure adequate compaction, and finishing techniques to assure satisfactory surface textures. Sample mix proportions are tabulated. A listing of references is provided covering proportioning, properties, refractory uses, shotcrete technology, and general information on SFRC.

Keywords: compacting; concrete construction; concrete finishing (fresh concrete); fiber reinforced concretes; mixing; mix proportioning; metal fibers; placing; specifications.

CONTENTS

CHAPTER 1 — GENERAL

1.1 — Scope

This guide covers specifying, mixing, placing, and finishing of steel fiber reinforced concrete (SFRC).

1.2 — Steel fiber reinforced concrete—General

Steel fiber reinforced concrete is a composite material made of hydraulic cements, fine or fine and coarse aggregate, and a dispersion of discontinuous, small, steel fibers. It may also contain pozzolans and additives commonly used with conventional concrete.

The addition of these fibers in amounts from 50 to 200 lb/yd³ (30 to 118 kg/m³)* can provide significant

*0.38 to 1.5 percent by volume (for normal weight concrete - 145 lb/ft³).

Alternative 1: The purchaser assumes responsibility for mixture proportions and specifies them including cement content, maximum allowable water content, and the type, name, and dosage of admixtures, if admixtures are to be used.

Alternative 2: The purchaser requires the concrete supplier to assume responsibility for selecting mixture proportions and specifies a minimum flexural or compressive strength.

Alternative 3: The purchaser requires the concrete supplier to assume responsibility for selecting mixture proportions, but with a minimum allowable cement content specified, along with the required minimum flexural or compressive strength and the type, name, and dosage of admixtures to be used.

For those projects electing to use Alternative 1, typical airfield paving mixtures are shown in Chapter 3 as examples of mixtures that have been used for paving and similar applications. Note that the amount of fiber to suit the particular application must be selected and specified. Adjustments to these mixtures, as may be required for workability, placeability, surface texture, yield, or other properties, should be made and evaluated during trial mixture or preconstruction testing, and final mixture proportions should be selected during the first day or so of actual construction.

CHAPTER 2 — MATERIALS

2.1 — General

Cement, pozzolans, aggregates, water, admixtures, reinforcing steel, and other conventional materials to be used for steel fiber reinforced concrete should conform to the same nationally recognized specifications used for conventional concrete. Since these specifications are named in other ACI publications, many are not repeated here. Titles for those that are referenced in this guide are shown in Chapter 8 — Information Sources.

2.2 — Aggregates

The fine aggregate should meet the gradation requirement given in ASTM C 33. The coarse aggregate should be ASTM C 33 size No. 8 or equivalent for nominal ⅜ in. (9 mm) maximum size aggregate mixtures and should be size No. 67 or equivalent for ¾ in. (19 mm) maximum size aggregate mixtures. Aggregate sizes larger than ¾ in. (19 mm) are not generally used in SFRC.

2.3 — Fibers

A specification for steel fibers (A 820) is being prepared by the ASTM but has not yet been published. In the interim, fibers are commonly specified by brand name or, for public works, by a short specification description which usually includes a minimum ultimate tensile strength (they are available from 50,000 psi up to 300,000 psi), a minimum desired aspect ratio ℓ/d, and any other desired features such as end anchorage provisions, deformations, or collating. The specifier should consult the fiber manufacturers for details.

Steel fibers should be clean, free of rust, oil, and deleterious materials. Steel fibers should have an aspect ratio, i.e., fiber length divided by diameter (or equivalent diameter,* in the case of nonround fibers), in the range of 30 to 100 for lengths of 0.5 to 2.5 in. (12.7 to 63.5 mm).

2.4 — Admixtures

- Calcium chloride should not be added to steel fiber reinforced concrete in excess of amounts permitted to be added to conventional structural concrete as shown in ACI 318-83, Table 4.5.4.
- Water-reducing admixtures are recommended. Both regular and high-range (superplasticizer) water reducers are suitable.
- Air-entraining admixtures are recommended for SFRC exposed to freezing and thawing conditions.

2.5 — Storage of fibers

Care should be taken to see that steel fibers are stored in a manner that will prevent their deterioration or the intrusion of moisture or foreign matter. If fibers deteriorate or become contaminated, they should not be used.

CHAPTER 3 — TYPICAL MIXTURE PROPORTIONS

3.1 — Mixture proportions

As with conventional concrete, SFRC mixtures employ a variety of mixture proportions depending upon the end use. They may be specially proportioned for a project or selected to be the same as a mixture used previously. In either case, they must be adjusted for yield, workability, and other factors as noted in Section 1.4.2.

A procedure for proportioning of SFRC mixtures with emphasis on good workability, is available.[9] Typical proportions that have been used for airfield paving mixtures are shown in Table 1. In addition, ACI 544.1R, Chapter 3, discusses SFRC mixtures and includes a table showing the range of proportions for normal weight fiber reinforced concrete.

CHAPTER 4 — FORMWORK AND REINFORCING STEEL

4.1 — Formwork

Design and construction of formwork should be done according to ACI 347. Normal weight SFRC with a fiber content up to two percent by volume has a density in the same range as normal weight conventional concrete — 144 to 150 lb/ft³ (2306 to 2403 kg/m³). The fibers in steel fiber reinforced concrete have a tendency to protrude from sharp corners of formed concrete. These may be hazardous to personnel. To minimize this sharp corners should be chamfered. Alternately, a rounded corner may be formed by applying a pressur

*The equivalent diameter of a fiber is the diameter of a round fiber having the same cross-sectional area A as the fiber in question; equivalent diameter $\sqrt{4A/\pi}$.

sensitive tape to the inside of sharp corners in the forms. On formed surfaces, use of a form vibrator will cause the fibers to back away from the form leaving them covered by about 1/8 in. (3 mm) of concrete. Formwork must be designed for the additional stress caused by the vibration. Consult ACI 347 for further information.

4.2 — Reinforcing steel

Fabricating and placing reinforcing steel should be in accordance with ACI 301. Steel fiber reinforced concrete is routinely used in conjunction with reinforcing steel.

CHAPTER 5 — BATCHING, MIXING, DELIVERY, AND SAMPLING

5.1 — General

Batching, mixing, delivery, and sampling of steel fiber reinforced concrete should be in accordance with ASTM C 94, Ready-Mixed Concrete, or applicable portions of ACI 304, Measuring, Mixing, Transporting, and Placing Concrete, as modified and supplemented by the following.

The contractor should supply appropriate equipment or develop a suitable technique for dispersing the fibers in the mixer free of fiber clumps. The equipment and/or method of adding the fibers to the mix should be reviewed and accepted by the project engineer before any placement of SFRC takes place.

The batching procedure is critical to obtaining a good blend of the fibers with the concrete. Several methods have previously been used with success, and information to assist the contractor in the choice of a suitable procedure may be obtained from fiber manufacturers. Any SFRC which is not properly batched and which develops dry clumps of fibers or a significant number of wet fiber balls (which include fibers and matrix) should be discarded and removed from the site.

The contractor should perform a full-scale trial batching, charging, and mixing operation with a minimum of 80 percent of the planned operational batch size at least eight days* prior to the first SFRC placement. The owner's engineer should observe the operation and recommend adjustments in the mixture proportions at the time to help obtain a workable mixture at a low water-cement ratio. Additional batches may be necessary to verify the mixture adjustments and plant efficiency. The contractor should conduct quality control tests for the trial batches and the owner may elect to cast test specimens for quality assurance. At the time of the test batch, the contactor should have on hand a working vibrator of the type to be used in the actual placements. The behavior of the trial batch under this vibration should be observed to provide guidance under actual construction operations.

Mixers generally should not be batched over 85 percent of their rated capacity[†] for SFRC. The mixing time should be sufficient to uniformly distribute the fibers in the mixture.

Table 1 — Steel fiber reinforced concrete mixtures used in the construction of airfield paving*

	Mixture Number 1, 3/8 in. (9 mm) aggregate with pozzolan or fly ash	Mixture Number 2, 3/4 in. (19 mm) aggregate with pozzolan or fly ash
Cement	500 lb/yd³ (296 kg/m³)	525 lb/yd³ (311 kg/m³)
Fly ash or pozzolan	235 lb/yd³ (139 kg/m³)	250 lb/yd³ (148 kg/m³)
Steel fibers[†]	•	•
Coarse aggregate[‡]		
[3/8 in. (9 mm) max]	1470 lb/yd³ (872 kg/m³)	
[3/4 in. (19 mm) max]		1330 lb/yd³ (789 kg/m³)
Sand	1370 lb/yd³ (812 kg/m³)	1440 lb/yd³ (854 kg/m³)
Water[§]	255 lb/yd³ (151 kg/m³)	283 lb/yd³ (168 kg/m³)
Additives		
Water reducers (normal or high range)	Per manufacturer's instructions.	
Air-entraining agent	Per manufacturer's recommendation for 6 percent air when subject to freezing and thawing conditions.	

*These mixture proportions are given as examples. The exact mixture proportions required to produce 1 yd³ (or 1 m³) for any given project will depend on a number of factors, such as the specific gravity of the materials and the water and air content of the mixture. Each mixture should be designed to yield correctly. These mixtures have been placed by slipform pavers.

[†]Fiber content of these airfield paving mixtures varied from 83 lb/yd³ to 140 lb/yd³ (49 to 83 kg/m³). Flexural strengths ranged from 1050 to 1100 psi (7.2 to 7.6 MPa) at 28 days.

[‡]Aggregates larger than 3/4 in. (19 mm) are not generally used in steel fiber concrete. If larger aggregates are used, the use of longer fibers should be considered.

§Water content varies depending upon amount and type of water reducer, workability achieved with the aggregates used, and other factors. Field adjust to optimum for strength and workability.

5.2 — Mixing

There are some important differences in mixing SFRC in a transit mixer or revolving drum mixer compared to conventional concrete. One of these is that to obtain good dispersion of the fibers and to prevent fiber clumping, the fibers should be added to a fluid mix.

Methods 1 and 2, which follow, describe procedures used to mix SFRC by adding the fibers to a fluid mix. These methods generally apply to uncollated, individual fibers. Fibers collated into bundles of about 30 fibers per bundle using a water-reactive glue may be dumped directly into a fluid mix as the last step (i.e., similar to Method 1 below) with little or no likelihood of fiber clumping. Also, certain types of individual fibers of large diameter, [e.g., 0.035 in. (0.9 mm) equivalent diameter half round fibers up to 2½ in. (63 mm) long] and conventional round or rectangular fibers with an aspect ratio less than 50 may generally be added to a fluid mix as the last step with little or no likelihood of fiber clumping.

Method 1: Add fibers last to transit mix truck

1. The wet mixture to be used is prepared first without the fibers. The slump of the concrete before fiber addition should be 2 to 3 in. (51 to 76 mm) greater than the final slump desired. Normally, the mixture would be prepared using the water-cement ratio found to give the best results and meeting the mixture design specifications for the job.

*To allow time to make appropriate 7-day tests.

[†]This figure may be raised to 100 percent if it can be shown that the equipment is capable of producing a uniformly mixed product.

2. With the mixer operating at normal charging speed, add the fibers as described in 3.

3. Add the individual fibers, clump-free (i.e., as a rain of individual fibers), to the mixer. A convenient way to do this is to dump the fibers through a 4 in. (100 mm) mesh screen into a hopper which opens onto a moving conveyor belt going to the mixer. It is important that no clumps be introduced; once a clump is introduced into the mixture, it will remain a clump. The drum must rotate fast enough to carry away the fibers as they enter the mixture, and the fibers should land on the mixture. With all fibers introduced into the mixer, it should be slowed to the rated mixing speed and mixed for approximately 30-40 revolutions.

Method 2: Add fibers to aggregate on a conveyor belt

In a plant set up to charge a central mixer or transit mixers, add the fibers by a shaker or through a hopper to the fine aggregate on a conveyor belt during aggregate addition and mix in the normal manner. This method does not require the same care as Method 1 as to where the fibers land in the mixer, but they should not be allowed to pile up and form clumps on their way to the mixer. If possible, the operator should stretch out the addition of aggregate so that fibers go in with the aggregate and not by themselves. A fiber feeder or shaker is useful in reducing the time for fiber handling and addition. Method 2 has been used for the majority of fibrous concrete projects where large quantities of concrete were mixed using bulk individual fibers.

5.3 — Causes of fiber clumping

The following listing of causes of fiber clumping may be useful in designing a plant or mixing sequence for fibrous concrete or correcting the problem in a mixing operation. Most fiber clumping occurs somewhere before the fibers get into the mixture. Once the fibers get into a mixture clump-free, they nearly always stay clump-free. This means that if clumps form, it's because fibers were added so that they fell on each other and they stacked up (in the mixer, on the belt, on the vanes, etc.). This normally happens when the fibers are added too fast at some point in the procedure. The mixer, whatever type, must carry the fibers away into the mixture as fast as they are added. Clumps can also form by hanging up on a rough loading chute at the back of a mixer truck. Fibers should not be allowed to pile up or slide down the vanes of a partially filled drum; this will form clumps.

Other causes of clumping are adding too many fibers to a mixture (more than about 2 percent by volume or even 1 percent of a fiber with a high aspect ratio); adding fibers too fast to a harsh mixture (mixture is not fluid enough or workable enough and the fibers don't get mixed in fast enough; therefore, they pile up on each other in the mixer); adding fibers first to the mixer (fibers have nothing to keep them apart, they fall on each other and form clumps); and using equipment with worn out mixing blades. The most common causes of wet fiber balls are overmixing and using a mixture with too much coarse aggregate (more than about 50 percent).

CHAPTER 6 — PLACING AND FINISHING

6.1 — General

Conventional concrete equipment is adequate for the placing and finishing of nearly all steel fiber reinforced concrete. Internal or external vibrators (including vibrating screeds) should always be used or the concrete will have excessive pockets of entrapped air voids. Even if a superplasticizer has been used, some vibration is needed around reinforcing steel to avoid reduction of bond to the bars.

The basic guide for placing concrete, ACI 304, should be used for placing and finishing SFRC along with the different techniques noted in the following.

6.2 — Workability and consistency measurements

Because of the unique properties of SFRC, workability measurements or slump requirements will be somewhat different from those of conventional concrete. Acceptable workability of SFRC should be determined by one of the following methods, and its use should be specified in the contractual documents.

6.2.1 *Time of flow through the inverted slump cone* — The inverted slump cone procedure (ASTM C 995) may be used to determine the workability of SFRC. This test apparatus consists of a conventional slump cone inverted, centered, and rigidly held by external supports so that the small end of the cone is 3 in. (75 mm) off the bottom of a one cubic foot yield bucket (ASTM C 29). The slump cone is loosely filled with an uncompacted concrete sample. The test uses a vibrator conforming to ASTM C 31 or C 192 with a 1 ± 1/8 in. (25 ± 3 mm) diameter probe. The probe of the operating vibrator is allowed to fall under its own weight through the concrete in the slump cone to the bottom of the bucket and its end is allowed to rest on the bottom of the bucket. The elapsed time from when the vibrator first makes contact with the concrete until the slump cone first becomes emptied is recorded as the inverted-slump-cone time. The inverted-slump-cone time for SFRC should preferably be not more than about 30 seconds nor less than about 10 seconds. These times may not suit all mixtures. Changes in fiber length and amount, cement content, sand content, air content, aggregate shape, and other factors may produce a different acceptable time. Also, the test is not applicable to concrete that flows freely through the cone.

6.2.2 *Slump test* — The slump test may be specified in the contractual documents to serve as a control test for consistency of SFRC from batch to batch. (In addition to the slump test described in ASTM C 143, it may be appropriate to perform the tests described in ASTM C 138, C 173, and C 231 also.)

In general, the slump for steel fiber reinforced concrete per ASTM C 143 should be at least 1 in. but not greater than 4 in. (25 mm to 100 mm). However, the same factors that influence inverted-slump-cone time also influence the slump. When these factors are changed, a different range may be acceptable. In any event the specified water-cement ratio should be maintained.

6.3 — Placing

Because of fibers, SFRC with the proper water-cement ratio appears very stiff and unworkable until subjected to vibration. Then it usually places very easily. The material tends to "hang together" and resist movement or compaction if an attempt is made to handle it without vibration. Batch plant operators and transit truck drivers must be instructed not to add additional water to the mixture based on its appearance and their experience with conventional concrete.

Water-cement ratios for fibrous mixtures must be carefully controlled. It is very easy to add unnecessary water to the mixture and lose many of the beneficial properties obtained from the addition of fibers. Ratios on the order of about 0.43 to 0.50 are normal. Paving mixtures and some special structural applications may benefit from less workable, but much higher quality concrete with the water-cement ratios in the range of 0.40 to 0.43. At the upper end of the water-cement spectrum, tests have shown that further addition of water causes an increase in slump without a change in workability under vibration. This water addition reduces the quality of the mixture without improving the placeability.

There are no special measures to take for placing SFRC around reinforcing steel except to use vibration to properly consolidate it. In a very thin wall or beam form, e.g., 4 in. (100 mm) or less, which also contains bars or mesh, placement of the concrete may be difficult, especially with longer fibers. This is similar to the difficulties of placing conventional mixtures with larger aggregate in thin, congested sections. When SFRC mixtures are used in congested areas, a ⅜ in. (9 mm) maximum aggregate size should be specified to reduce placing difficulties.

6.4 — Transporting and handling equipment

Transporting and placing of SFRC can be accomplished with most conventional equipment that is properly designed, maintained, and clean.

6.4.1 *Transit trucks* — Discharging from transit trucks is usually accomplished with little trouble. Too stiff a mixture or a truck in poor condition will prevent the mixture from easily discharging from the back of the drum onto the chute. A well-proportioned mixture usually just barely slides down the chute by itself and may need to be pushed by the truck operator. When an especially stiff mixture is used, the truck can be driven up on blocks or a ramp to help discharging.

6.4.2 *Concrete buckets* — Concrete buckets should have steep hopper slopes, be clean and smooth inside, and have a minimum gate opening dimension of 12 in. (300 mm). The fibers will bridge smaller gate openings and the mix will not fall out of its own weight. A remedy for bridging and an aid to placement is to provide a vibrator at the bucket when discharging. To facilitate placement of especially stiff mixtures, a form vibrator can be attached to the side of the bucket and activated when the gate is opened. Another procedure is to weld pieces of pipe to the bucket exterior. Internal vibrators can then be placed into the pipes to assist in emptying the bucket.

6.4.3 *Pumping* — Pumping has been used to transport SFRC on a number of projects. A good fiber mixture generally has proportions of sand and admixtures which make it well-suited for pumping. Gradations suited to SFRC are also compatible with pumping. Although a mixture may appear stiff and unworkable, it may pump surprisingly well. Because of its composition, a SFRC mixture will move through the line without slugs and has been reported to pump more easily and more trouble-free than conventional concrete. Some important points about pumping SFRC are (1) use a pump capable of handling the volume and pressures needed; (2) use a large diameter line, preferably at least 6 in. (150 mm); (3) avoid flexible hose if possible; (4) provide a screen over the pump hopper to prevent any fiber clumps from entering the line. About a 2 × 3 in. (50 × 75 mm) mesh is adequate; and (5) do not try to pump a fibrous mix that is too wet. Pump pressures can cause the fluid paste and fine mortar to squeeze out ahead of the rest of the mixture, resulting in a mat of fibers and coarse aggregate without mortar. It must be noted that this is the result of a mixture that is too wet, not too dry. The same type of plugging can occur with conventional concrete with the plug consisting of coarse aggregate devoid of paste and fine mortar. Additional information on pumping is available in ACI 304.2R. Reference 10 describes proportioning of SFRC mixes for pumping.

6.5 — Finishing

Steel fiber reinforced concrete can be finished with conventional equipment, but minor refinements in techniques and workmanship are needed. For flat formed surfaces, normally no special attention is needed. The surface will normally be smooth and not show fibers when the forms are stripped. If chamfers or rounds have been provided at the edges and in corners the ends of fibers will not protrude at these points when forms are stripped. To provide added compaction and bury surface fibers, open slab surfaces should first be struck off with a vibrating screed. The screed should have slightly rounded edges and preferably should be metal. In areas where a screed is not practical, a jitterbug* can be used for compaction and to establish rough grade control. Magnesium floats can be used to establish a surface and close up any tears or open areas which are caused by the screed. Wood floats tend to tear the surface and should not be used.

Throughout all finishing operations, care must be taken not to overwork the surface. Overworking will bring excessive fines to the surface and may result in

*A grate tamper that forces aggregate and fibers below the surface.

crazing which normally shows up after the curing period. If bleeding occurs or excessive fines are at the surface, the material should be screeded off and discarded.

After completion of any float work, the surface should be left until it can be worked further without damage. This is usually about the time of initial set. Where a careful finish is not required for appearance or exact tolerance, no further work is needed after floating. If a texture is required, a broom or roller can be used prior to initial set. Burlap drags should not be used because they will lift up the fibers and tear up the surface. When additional finishing is needed, the next step should be done with magnesium floats. Power equipment or hand equipment may be used. When done by hand, the float should be held flat and not on edge. It should be moved with a sawing motion (short, quick, back-and-forth movements) as it is drawn across the surface. The magnesium float can be used to obtain a nearly perfect, flat surface, bury or cover all the fibers, and leave a slight texture. This can be followed by hard steel troweling if a smooth surface is desired. The trowel must be kept flat or the edge will cause fibers to spring out of the surface. Using these techniques, some excellent finishes of SFRC have been obtained. An example of this is at Tarbela Dam where a curved spillway invert and flat stilling basin floor were completed to close tolerances.

Slipform pavers have been used on several projects, such as airport runways and taxiways, with excellent results.

The proper time to execute a broom finish following a screed finish or paving machine finish is just prior to application of curing compounds when the water sheen has practically disappeared.

6.6 — Hot and cold weather requirements

Placement of steel fiber reinforced concrete should be done according to the recommendations of ACI 305R for hot weather and ACI 306R for cold weather.

6.7 — Repair of defects

The repair of defects such as voids and honeycombing is done much the same as for plain concrete. However, if removal of some SFRC is required, the removal operation will be significantly more difficult because of the greater toughness of SFRC.

Removal by jackhammers is hindered because the material does not fracture easily. Sawing is a more effective method of cutting or removing steel fiber reinforced concrete.

6.8 — Contraction joints

Contraction joints in slabs are more easily made if they are sawed rather than cast or formed. The sawing can be done shortly after final set. At joints where it is desired to have a controlled shrinkage crack occur below the sawed portion of the joint, it has been found that the saw cut should extend from one-half to two-thirds of the way through the slab. If it does not, the higher tensile strength of the SFRC tends to preven cracking at the joint and random cracking occurs else where in the slab. A joint sealing compound should b used to seal the sawed joint to prevent water infiltra tion to the subgrade, and to prevent the corrosion c those fibers and fiber ends that become exposed i the saw cut and the crack below.

CHAPTER 7 — CURING AND PROTECTION

7.1 — General

Curing of steel fiber reinforced concrete and protec tion from freezing or excessively hot or cold tempera tures should be done the same as for conventional cor crete. One aspect deserves special attention. Sinc SFRC is often placed in thin sections, as overlays fc example, and has a high cement content, it is partict larly vulnerable to plastic shrinkage cracking. This wi occur on warm days where it is exposed to direct sun c a breeze. Such placements must be shaded from the su and sheltered from the wind to prevent this type c damage.

CHAPTER 8 — INFORMATION SOURCES

8.1 — Specified and/or recommended references

The standards of the American Society for Testir and Materials and the standards and reports of th American Concrete Institute referred to in this repo are listed below with their serial designation, includir the year of adoption or revision. The standards and re ports listed were the latest editions at the time this r port was prepared. Since some of these publications a revised frequently, generally in minor details only, th user of this report should check directly with the spor soring group to refer to the latest edition.

American Concrete Institute

ACI 301-72 (Revised 1982)	Specifications for Structural Co crete for Buildings
ACI 304-73 (Reaffirmed 1978)	Recommended Practice for Me suring, Mixing, Transporting, ar Placing Concrete
ACI 304.2R-71 (Revised 1982)	Placing Concrete by Pumpir Methods
ACI 305R-77	Hot Weather Concreting
ACI 306R-78	Cold Weather Concreting
ACI 318-83	Building Code Requirements f Reinforced Concrete
ACI 347-78	Recommended Practice for Co crete Formwork
ACI 544.1R-82	State-of-the-Art Report on Fib Reinforced Concrete
ACI 544.2R-78	Measurement of Properties of Fib Reinforced Concrete

American Society for Testing and Materials

A 820 (To be published)	Standard Specification for Steel I bers for Fiber Reinforced Concre
C 29-78	Standard Test Methods for Ur Weight and Voids in Aggregate

C 31-83	Standard Method for Making and Curing Concrete Test Specimens in the Field
C 33 82	Standard Specification for Concrete Aggregates
C 78-75 (Reapproved 1982)	Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)
C 94-81	Standard Specification for Ready-Mixed Concrete
C 138-81	Standard Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete
C 143-78	Standard Test Method for Slump for Portland Cement Concrete
C 173-78	Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method
A 192-81	Standard Method of Making and Curing Concrete Test Specimens in the Laboratory
C 231-82	Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method
C 995-83	Time of Flow of Fiber-Reinforced Concrete Through Inverted Slump Cone

The above publications may be obtained from the following organizations:

American Concrete Institute
P.O. Box 19150
Detroit, MI 48219

American Society for Testing and Materials
1916 Race Street
Philadelphia, PA 19103

8.2 — Cited references

General

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2. "Fibre Concrete Materials: A Report Prepared by RILEM Technical Committee 19-FRC," *Materials and Structures/Research and Testing* (RILEM, Paris), V. 10, No. 56, Mar.-Apr. 1977, pp. 103 120.

Properties of steel fiber concrete

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Refractory concrete

6. Lankard, D. R., "Steel Fiber Reinforced Refractory Concrete," *Refractory Concrete*, SP-57, American Concrete Institute, Detroit 1978, pp. 241-263.

7. Hackman, L. E., "Application of Steel Fiber to Refractory Reinforcement," *Proceedings*, Symposium on Fibrous Concrete (Ci80 London, 1980), The Construction Press, Lancaster, 1980, pp. 137 152.

Shotcrete

8. Henager, Charles H., "Steel Fibrous Shotcrete: A Summary o the State-of-the-Art," *Concrete International: Design & Construction*, V. 3, No. 1, Jan. 1981, pp. 50-58.

Proportioning of steel fiber concrete mixes

9. Schrader, Ernest K., and Munch, Anthony V., "Deck Slab Repaired by Fibrous Concrete Overlay," *Proceedings*, ASCE, V. 102 CO1, Mar. 1976, pp. 179-196. (Includes Appendix: Mix Design Procedures.)

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ACI COMMITTEE 544
Fiber Reinforced Concrete

Erosion of Concrete in Hydraulic Structures

Reported by ACI Committee 210

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This report outlines the causes, control, maintenance, and repair of erosion in hydraulic structures. Such erosion occurs from three major causes: cavitation, abrasion, and chemical attack. Design parameters, materials selection and quality, environmental factors, and other issues affecting the performance of concrete are discussed.

Evidence exists to suggest that given the operating characteristics and conditions to which a hydraulic structure will be subjected, it can be designed to mitigate future erosion of the concrete. All too often, however, operational factors change or are not clearly known and hence erosion of concrete surfaces occurs and repairs must follow. This report briefly treats the subject of concrete erosion and repair and provides numerous references to detailed treatment of the subject.

Keywords: abrasion; abrasion resistance; aeration; cavitation; chemical attack; concrete dams; concrete pipes; corrosion; corrosion resistance; deterioration; erosion; grinding (material removal); high-strength concretes; hydraulic structures; maintenance; penstocks; pipe linings; pipes (tubes); pitting; polymer concrete; renovating; repairs; spillways; tolerances (mechanics); wear.

CONTENTS

Committee 210 recognizes with thanks the contributions of Jeanette M. Ballentine, J. Floyd Best, Gary R. Mass, William D. McEwen, Myron B. Petrowsky, Melton J. Stegall, and Stephen B. Tatro.

PART I — CAUSES OF EROSION

CHAPTER 1 — INTRODUCTION

Erosion is defined in this report as the progressive disintegration of a solid by cavitation, abrasion, or chemical action. This report is concerned with cavitation erosion resulting from the collapse of vapor bubbles formed by pressure changes within a high-velocity water flow; with the abrasion erosion of concrete in hydraulic structures caused by water-transported silt, sand, gravel, ice, or debris; and with disintegration of the concrete by chemical attack. Other types of concrete deterioration are outside the scope of this report.

Ordinarily, concrete in properly designed, constructed, used, and maintained hydraulic structures will undergo years of erosion-free service. However, for a variety of reasons including inadequate design or construction, or operational and environmental changes, erosion does occur in hydraulic structures. This report deals with three major aspects of such concrete erosion:

Part 1 discusses the three major causes of concrete erosion in hydraulic structures: cavitation, abrasion, and chemical attack.

Part 2 discusses the options available to the designer and user to control concrete erosion in hydraulic structures.

Part 3 discusses the evaluation of erosion problems and provides information on repair techniques. Part 3 is not comprehensive, and is intended as a guide for the selection of a repair method and material.

CHAPTER 2 — EROSION BY CAVITATION

2.1 — Mechanism of cavitation

Cavitation is the formation of bubbles or cavities in a liquid. In hydraulic structures, the liquid is water, and the cavities are filled with water vapor and air. The cavities form where the local pressure drops to a value that will cause the water to vaporize at its ambient temperature. Fig. 2.1 shows examples of concrete surface irregularities which can trigger formation of these cavities. The pressure drop caused by these irregularities is generally abrupt and is caused by local high velocities and curved streamlines. Cavities often begin to form near curves or offsets in a flow boundary or at the centers of vortices.

When the geometry of flow boundaries causes streamlines to curve or converge, the pressure will drop in the direction toward the center of curvature or in the direction along the converging streamlines. For example, Fig. 2.2 shows a tunnel contraction in which a cloud of cavities could start to form at Point c and then collapse at Point d. The velocity near Point c is much higher than the average velocity in the tunnel upstream, and the streamlines near Point c are curved. Thus, for proper values of flow rate and tunnel pressure at 0, the local pressure near Point c will drop to the vapor pressure of water and cavities will occur. Cavitation damage is produced when the vapor cavities collapse. The collapses that occur near Point d produce very high instantaneous pressures that impact on the

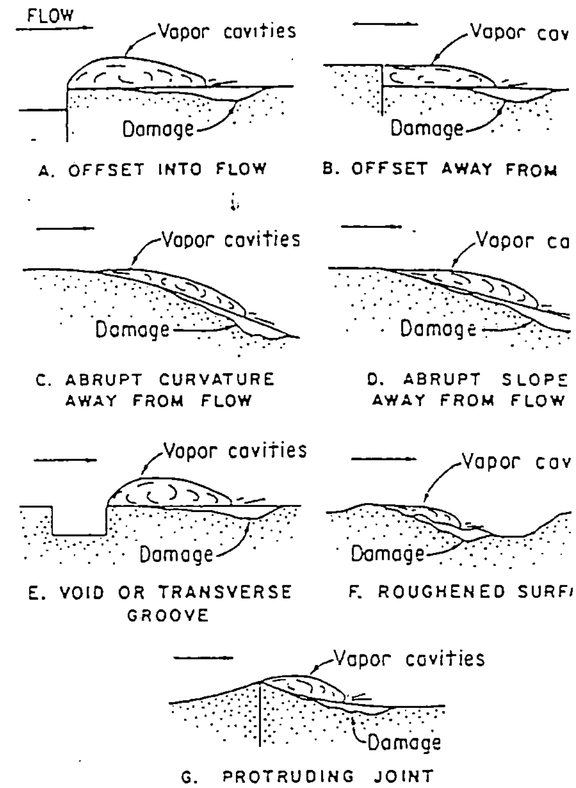


Fig. 2.1 — Cavitation situations at surface irreg ties

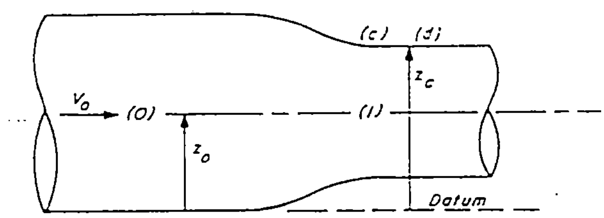


Fig. 2.2 — Tunnel contraction

boundary surfaces and cause pitting, noise, and v tion. Pitting by cavitation is readily distinguished the worn appearance caused by abrasion because tation pits cut around the harder coarse aggregate ticles and have irregular and rough edges.

2.2 — Cavitation index

The cavitation index is a dimensionless measure to characterize the susceptibility of a system to tate. Fig. 2.2 illustrates the concept of the cavit index. In such a system, the critical location for tation is at Point c.

The static fluid pressure at Location 1 will be

$$p_c + \gamma (z_c - z_0)$$

where p_c is the absolute static pressure at Point c; the specific weight of the fluid (weight per unit

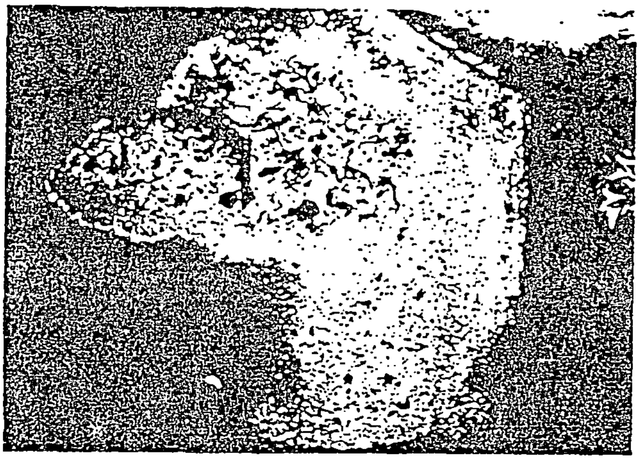


Fig. 2.3 — Cavitation erosion of intake wall of a navigation lock at point of tunnel contraction

ume); z_c is the elevation at Point c; and z_0 is the elevation at 0.

The pressure drop in the fluid as it moves along a streamline from the reference Location 0 to Location 1 will be

$$p_0 - [p_v + \gamma (z_c - z_0)]$$

where p_0 is the static pressure at 0.

The cavitation index normalizes this pressure drop to the dynamic pressure $\frac{1}{2} \rho V_0^2$

$$\sigma = \frac{p_0 - [p_v + \gamma (z_c - z_0)]}{\frac{1}{2} \rho V_0^2} \qquad (2\text{-}1)$$

where ρ is the density of the fluid (mass per unit volume) and V_0 is the fluid velocity at 0.

Readers familiar with the field of fluid mechanics may recognize the cavitation index as a special form of the Euler number or pressure coefficient, a matter discussed in Rouse (1978).

If cavitation is just beginning and there is a bubble of vapor at Point c, the pressure in the fluid adjacent to the bubble is approximately the pressure within the bubble, which is the vapor pressure p_v of the fluid at the fluid's temperature.

Therefore, the pressure drop along the streamline from 0 to 1 required to produce cavitation at the crown is

$$p_0 - [p_v + \gamma (z_c - z_0)]$$

and the cavitation index at the condition of incipient cavitation is

$$\sigma_c = \frac{p_0 - [p_v + \gamma (z_c - z_0)]}{\frac{1}{2} \rho V_0^2} \qquad (2\text{-}2)$$

It can be deduced from fluid mechanics considerations (Knapp, Daily, and Hammitt 1970) — and confirmed experimentally — that in a given system cavitation will begin at a specific σ_c, no matter which combination of pressure and velocity yields that σ_c.

If the system operates at a σ above σ_c, the sy does not cavitate. If σ is below σ_c, the lower the v of σ, the more severe the cavitation action in a g system. Therefore, the designer should insure tha operating σ is safely above σ_c for the system's cr location.

Actual values of σ_c for different systems differ m edly, depending on the shape of flow passages shape of objects fixed in the flow, and the loca where reference pressure and velocity are measure

For a smooth surface with slight changes of slo the direction of flow, the value of σ_c may be below For systems that produce strong vortices, σ_c ma ceed 10. Values of σ_c for various geometries are g in Chapter 5. Falvey (1982) provides additional in mation on predicting cavitation in spillways.

Since, in theory, a system having a given geon will have a certain σ_c despite differences in scale, σ useful concept in model studies. Tullis (1981) desc modeling of cavitation in closed circuit flow. Ca tion considerations (such as surface tension) in sc from model to prototype are discussed in Knapp, D and Hammitt (1970) and Arndt (1981).

2.3 — Cavitation damage

Cavitation bubbles will grow and travel with flowing water to an area where the pressure field cause collapse. Cavitation damage can begin at point. When a cavitation bubble collapses or impl close to or against a solid surface, an extremely pressure is generated, which acts on an infinites area of the surface for a very short time perioc succession of these high-energy impacts will damag most any solid material. Tests on soft metal show tial cavitation damage in the form of tiny craters. vanced stages of damage show an extremely rc honeycomb texture with some holes that penetrate thickness of the metal. This type of pitting often oc in pump impellers and marine propellers.

The progression of cavitation erosion in concre not as well documented as it is in metals. For l classes of material, however, the erosion progre rapidly after an initial period of exposure slig roughens the surface with tiny craters or pits. Pos explanations are that: a) the material immediately neath the surface is more vulnerable to attack; b) cavitation impacts are focused by the geometry of pits themselves; or c) the structure of the material been weakened by repeated loading (fatigue). In event, the photograph in Fig. 2.3 clearly shows a dency for the erosion to follow the mortar matrix undermine the aggregate. Severe cavitation damage typically form a Christmas-tree configuration on s way chute surfaces downstream from the point of gin as shown in Fig. 2.4.

Microfissures in the surface and between the mo and coarse aggregate are believed to contribute to itation damage. Compression waves in the water fills such interstices may produce tensile stresses w cause microcracks to propagate. Subsequent com

Fig. 2.4 — "Christmas tree" configuration of cavitation damage on a high-head tunnel surface

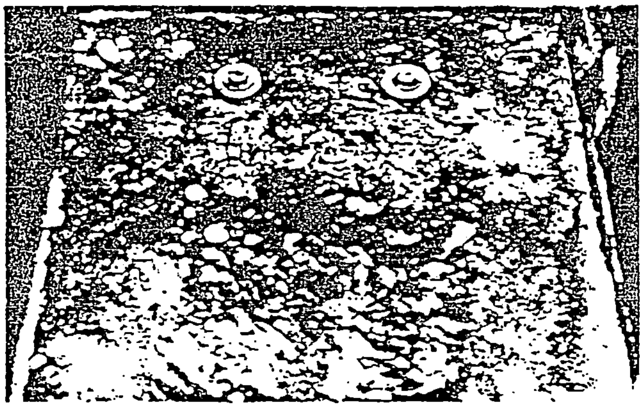


Fig. 2.5 — Concrete test slab featuring cavitation producing devices

sion waves can then loosen pieces of the material. The simultaneous collapse of all of the cavities in a large cloud, or the supposedly slower collapse of a large vortex, quite probably is capable of suddenly exerting more than 100 atmospheres of pressure on an area of many square inches. Loud noise and structural vibration attest to the violence of impact. The elastic rebounds from a sequence of such blows may cause and propagate cracks and other damage, causing chunks of material to break loose.

Fig. 2.5 shows the progress of erosion of concrete downstream from two protruding bolts used to generate cavitation. The tests were made at a test facility located at Detroit Dam, Oregon. Fig. 2.6 shows cavitation damage on test panels after 47 hours of exposure to high-velocity flow. A large amount of cavitation

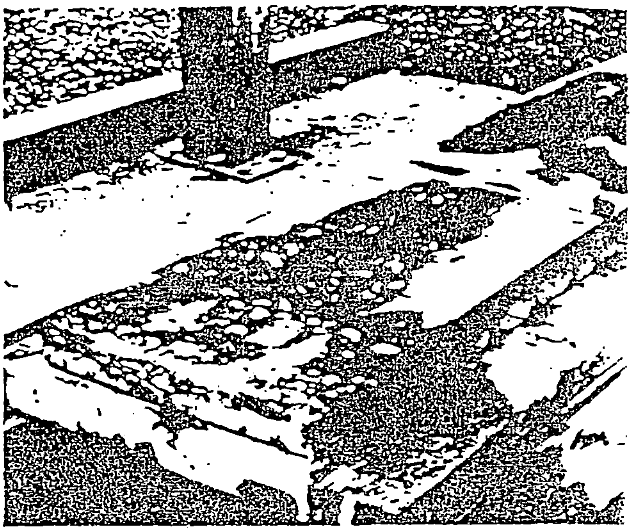


Fig. 2.6 — Cavitation erosion pattern after 47 hours of testing at a 240 ft velocity head

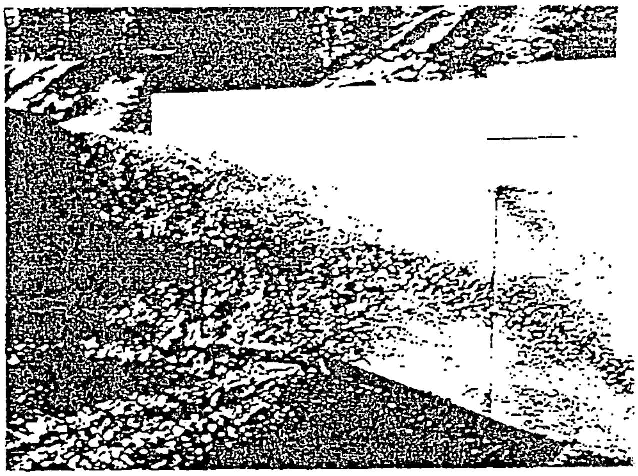


Fig. 2.7 — Cavitation erosion of discharge outlet training wall and flip bucket

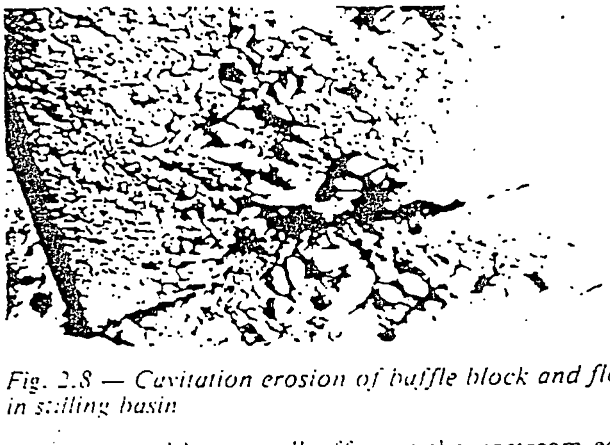


Fig. 2.8 — Cavitation erosion of baffle block and floc in stilling basin

erosion caused by a small offset at the upstream edg of the test slab is evident.

Fig. 2.7 shows severe cavitation damage that oc curred to the flip bucket and training walls of an outle structure at Lucky Peak Dam, Idaho. In this case, wa ter velocities of 120 ft/sec (37 m/sec) passed through a gate structure into an open outlet manifold, part o which is shown here. Fig. 2.8 shows cavitation damag

Fig. 3.3 — Typical debris resulting from abrasion erosion of concrete

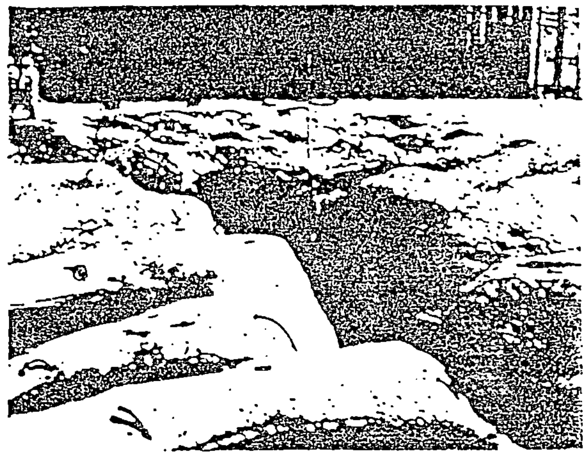


Fig. 3.4 — Erosion of stilling basin floor slab, [illegible] shak Dam

Fig. 3.5 — Abrasion erosion damage to stilling [illegible] Nolin Dam

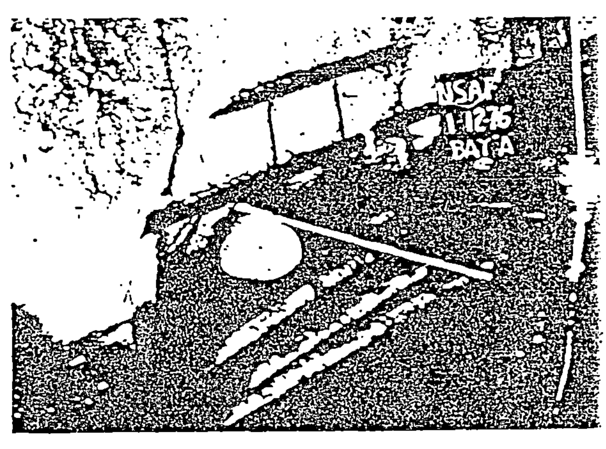


Fig. 3.6 — Abrasion erosion damage to discharge [illegible] eral, Upper St. Anthony Falls Lock

3.2 — Stilling basin damage

A typical stilling basin design includes a downstream sill from 3 to 20 ft (1 to 6 m) high intended to create a permanent pool to aid in energy dissipation of high-velocity flows. Unfortunately, in many cases these pools also trap rocks and debris (Fig. 3.3). The stilling basins at Libby and Dworshak Dams, high-head hydroelectric structures, were eroded to maximum depths of approximately 6 and 10 ft (2 and 3 m), respectively. In the latter case, nearly 2000 yd^3 (1530 m^3) of concrete and bedrock were eroded from the stilling basin (Fig. 3.4). Impact forces associated with turbulent flows carrying large rocks and boulders contribute to the surface damage of concrete.

There are many cases where the concrete in outlet works stilling basins of low-head structures has also exhibited abrasion erosion. Chute blocks and baffles within the basin are particularly susceptible to abrasion erosion by direct impact of waterborne materials. There also have been several cases where baffle blocks connected to the basin training walls have generated eddy currents behind these baffles, resulting in significant localized damage to the stilling basin walls and floor slab, as shown in Fig. 3.5

In most cases, abrasion erosion damage in stilling basins has been the result of one or more of the following: a) construction diversion flows through constricted portions of the stilling basin; b) eddy currents created by diversion flows or powerhouse discharges adjacent to the basin; c) construction activities in the vicinity of the basin, particularly those involving cofferdams; d) nonsymmetrical discharges into the basin; e) separation of flow and eddy action within the basin sufficient to transport riprap from the exit channel into the basin; f) failure to clean basins after completion of construction work; and g) topography of the outflow channel (McDonald 1980).

3.3 — Navigation lock damage

Hydraulic structures other than spillways are also subject to abrasion erosion damage. When Upper St. Anthony Falls navigation lock was dewatered to repair a damaged miter gate, an examination of the filling-emptying laterals and discharge laterals revealed [illegible]siderable abrasion erosion (Fig. 3.6). This erosion [illegible] the concrete to maximum depths of 23 in. (580 [illegible] was caused by rocks up to 18 in. (460 mm) in diam[illegible] which had entered the laterals, apparently during [illegible]charge of the flood of record through the lock

Fig. 3.1 — Abrasion damage to concrete baffle blocks and floor area in Yellowtail Diversion Dam sluiceway, Montana

to the side of a baffle block and the floor in the stilling basin at Yellowtail Afterbay Dam, Montana.

Once erosion has begun, the rate of erosion may be expected to increase because protruding pieces of aggregate become new generators of vapor cavities. In fact, a cavity cloud often is caused by the change in direction of the boundary at the downstream rim of an eroded depression. Collapse of this cloud farther downstream starts a new depression, and so on, as indicated in Fig. 2.4.

Once cavitation damage has substantially altered the flow regime, other mechanisms then begin to act on the surface. These other mechanisms include high water velocities striking the irregular surface and mechanical failure due to vibrating reinforcing steel. Significant amounts of material may be removed by these ad forces, thereby accelerating failure of the struct This sequence of cavitation damage followed by h impact damage from the moving water was clearly dent in the 1983 spillway tunnel failure at Glen Can Dam, Arizona.

CHAPTER 3 — EROSION BY ABRASION

3.1 — General

Abrasion erosion damage results from the abra effects of waterborne silt, sand, gravel, rocks, other debris being circulated over a concrete sur during operation of a hydraulic structure. Abra erosion is readily recognized by the smooth, worn pearing concrete surface, which is distinguished f the small holes and pits formed by cavitation eros as can be compared in Fig. 2.8 and 3.1. Spill aprons, stilling basins, sluiceways, and tunnel lin are particularly susceptible to abrasion erosion.

The rate of erosion is dependent on a number of tors including the size, shape, quantity, and hardne particles being transported, the velocity of the w and the quality of the concrete. While high-qu concrete is capable of resisting high water velocitie many years with little or no damage, the concrete not withstand the abrasive action of debris grindin repeatedly impacting on its surface. In such c abrasion erosion ranging in depth from a few inch several feet can result depending on the flow cc tions. Fig. 3.2 shows the relationship between fl bottom velocity and the size of particles which tha locity can transport.

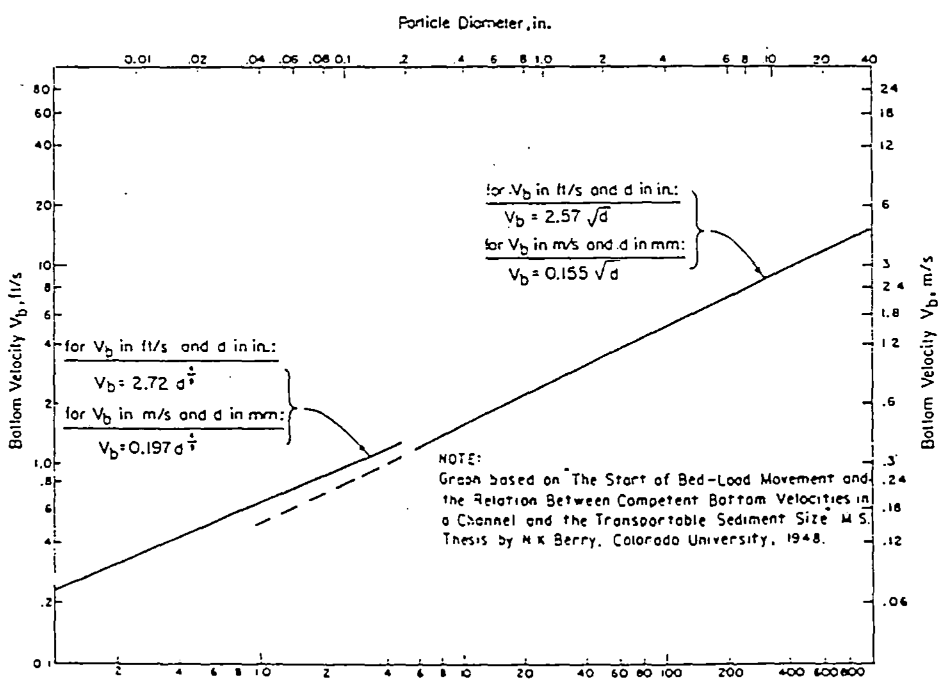


Fig. 3.2 — Bottom velocity versus transported sediment size

ber. Subsequent filling and emptying of the lock during normal operation agitated those rocks, causing them to erode the concrete by grinding.

3.4 — Tunnel lining damage

Concrete tunnel linings are susceptible to abrasion erosion damage, particularly when the water carries large quantities of sand, gravel, rocks, and other debris. There have been many instances where the concrete in both temporary and permanent diversion tunnels has experienced abrasion erosion damage. Generally, the tunnel floor or invert is the most heavily damaged. Wagner (1967) has described the performance of Glen Canyon Dam diversion tunnel outlets.

CHAPTER 4 — EROSION BY CHEMICAL ATTACK

4.1 — Sources of chemical attack

The compounds present in hardened portland cement are attacked by water and many salt and acid solutions; fortunately, in most hydraulic structures, the deleterious action on an impermeable mass of hardened portland cement concrete is so slow it is unimportant. However, there are exposures to chemical attack which can become serious and accelerate deterioration and erosion of the concrete at a very rapid rate.

Acidic environments can result in deterioration of exposed concrete surfaces. The acidic environment may range from low acid concentrations found in mineral-free water to high acid concentrations found in many processing plants. Alkali environments can also cause concrete deterioration. In the presence of moisture, alkali soils containing sulfates of magnesium, sodium, and calcium attack concrete, forming chemical compounds which imbibe water and swell, and can damage the concrete.

4.2 — Erosion by mineral-free water

Hydrated lime is one of the compounds formed when cement and water combine. It is readily dissolved by water and more aggressively dissolved by pure mineral-free water, found in some mountain streams. Dissolved carbon dioxide is contained in some fresh waters in sufficient quantity to make the water slightly acidic and add to its aggressiveness. Scandinavian countries have reported serious attacks by fresh water, both on exposed concrete surfaces and interior surfaces of conduits where porosity or cracks have provided access. In the United States, there are many instances where the surface of the concrete has been etched by fresh water flowing over it, but serious damage from this cause is uncommon (Holland et al. 1980). This etching is particularly evident at hydraulic structures carrying runoff from high mountain streams in the Rocky Mountains and the Cascade Mountains of the central and western United States. A survey (ICOLD 1951) of the chemical composition of raw water in many reservoirs throughout the United States indicates a nearly neutral acid-alkaline balance (pH) for most of these waters.

4.3 — Erosion by miscellaneous causes

4.3.1 *Acidic environments* — Decaying vegeta the most frequent source of acidity in natural w Decomposition of certain minerals may be a sou acidity in some localities. Running water that has as low as 6.5 will leach lime from concrete, reduc strength and making it more porous and less re to freezing and thawing and other chemical attacl amount of lime leached from concrete is a funct the area exposed and the volume of concrete. small-diameter drains will deteriorate in a few when exposed to mildly acidic waters, whereas pipe and massive structures will not be damaged icantly for many years under the same exposure vided the cover over the reinforcing steel meets n design standards. Waters flowing from peat bed have a pH as low as 5.

Acid of this strength will aggressively attack crete, and for this reason, when conveyance ground water are being designed, the aggressiven the water should be tested to determine its comp ity with the concrete. This is particularly true in sure conduits.

4.3.2 *Bacterial action* — Most of the literatu dressing the problem of deterioration of concre sulting from bacterial action has evolved because great impact of this corrosive mechanism on co sewer systems. This is a serious problem which, a don and Beardsley (1958) observed, occurs more ily in warm climates such as California, USA; tralia; and South Africa. This problem also occu the terminus of long pumped sewage force mains northern climates (Pomeroy 1974).

Sulfur-reducing bacteria belong to the genus of teria that derives the energy for its life processes the reduction of some element other than carbon, as nitrogen, sulfur, or iron (Rigdon and Bear 1958). Some of these bacteria are able to reduc sulfates that are present in natural waters and pr hydrogen sulfide as a waste product. These bacter stated by Wetzel (1975), are anaerobic.

Another group of bacteria takes the reduced s and oxidizes it back so that sulfuric acid is formed genus *Thiobacillus* is the sulfur-oxidizing bacteria is most destructive to concrete. It has a remarkabl erance to acid. Concentrations of sulfuric acid as as 5 percent do not completely inhibit its activity.

Sulfur-oxidizing bacteria are likely to be f wherever warmth, moisture, and reduced compo of sulfur are present. Generally, a free water surf required, in combination with low dissolved oxyg the sewage and low velocities that permit the build scum on the walls of a pipe in which the anaerobic fur-reducing bacteria can thrive. Certain condi must prevail before the bacteria can produce hydr sulfide from sulfate-rich water. Sufficient mois must be present to prevent the desiccation of the teria. There must be adequate supplies of hydr sulfide, carbon dioxide, nitrogen compounds, and ygen. In addition, soluble compounds of phosph iron, and other trace elements must be present in the moisture film.

Newly made concrete has a strongly alkaline surface with a pH of about 12. No species of sulfur bacteria can live in such a strong alkaline environment. Therefore, the concrete is temporarily free from bacterially induced corrosion. Natural carbonation of the free lime by the carbon dioxide in the air slowly drops the pH of the concrete surface to 9 or less. At this level of alkalinity, the sulfur bacteria *Thiobacillus thioparus,* using hydrogen sulfide as the substrate, generate thiosulfuric and polythionic acid. The pH of the surface moisture steadily declines, and at a pH of about 5, *Thiobacillus concretivorus* begins to proliferate and produce high concentrations of sulfuric acid, dropping the pH to a level of 2 or less. The destructive mechanism in the corrosion of the concrete is the aggressive effect of the sulfate ions on the calcium aluminates in the cement paste.

The main concrete corrosion problem in a sewer, therefore, is chemical attack by this sulfuric acid which accumulates in the crown of the sewer. Information is available which may enable the engineer to design, construct, and operate a sewer so that the development of sulfuric acid is reduced (Pomeroy 1974, ASCE-WPCF Joint Task Force 1982; ACPA 1981).

PART 2 — CONTROL OF EROSION

CHAPTER 5 — CONTROL OF CAVITATION EROSION

5.1 — Hydraulic design principles

In Chapter 2, Section 2.2, the cavitation index σ was defined by Eq. (2-1). When the value of σ at which cavitation damage begins is known, a designer can calculate velocity and pressure combinations that will avoid trouble. To produce a safe design, the object is to insure that the actual operating pressures and velocities will produce a value of σ greater than the value at which damage begins.

A good way to avoid cavitation erosion is to make σ large by keeping the pressure p_0 high, and the velocity V_0 low. For example, deeply submerged baffle piers in a stilling basin downstream from a low spillway are unlikely to be damaged by cavitation because both of these conditions are satisfied. This situation is illustrated in Fig. 5.1. The following example illustrates how σ is calculated for this case. From model studies, the mean prototype velocity at 0, immediately upstream from the baffle block, is found to be 30 ft/sec (9.1 m/sec), and the "minimum" prototype gage pressure, exceeded 90 percent of the time, is 7.1 psi (49 kPa). The barometric pressure for the prototype location is estimated to be 13.9 psi (95.8 kPa), so that the absolute pressure at 0, 6.6 ft (2.0 m) above Location 1, becomes

$$p_0 = 7.1 + 13.9 - \frac{(6.6 \times 62.4)}{144\ \text{in.}^2/\text{ft}^2} = 18.1\ \text{psi}$$

Given that

$$p_v = 0.3\ \text{psi}$$
$$\rho = 1.94\ \frac{\text{lb}\cdot\text{sec}^2}{\text{ft}^4}$$

and

$$z_c = z_0$$

it follows that

$$\sigma = \frac{p_0 - p_v}{\frac{1}{2}\ \rho\ V_0^2} \qquad (5\text{-}1$$

and

$$\sigma = \frac{(18.1 - 0.3)\ (144\ \text{in.}^2/\text{ft}^2)\ (32.2\ \text{ft/sec}^2)}{\frac{1}{2}(62.4)\ (30)^2} = 2.9$$

(In SI metric units

$$p_0 = 49 + 95.8 - \left(2.0 \times 9.81\ \frac{\text{kgf/m}^2}{\text{Pa}}\right) = 125\ \text{kPa}$$

Then, given that $p_v = 2.1$ kPa, $\rho = 10^3$ kg/m³, and $z_c = z_0$

$$\sigma = \frac{(125 - 2.1)\ (1000)}{\frac{1}{2}\ (1000)\ (9.1)^2} = 2.9).$$

This value of σ is well above the accepted damage value of 2.3 for this shape of sharp-edged pier (Galperin et al. 1977). Hence, cavitation damage is unlikely in the prototype. A second, equally effective procedure to avoid cavitation is to use boundary shapes and tolerances characterized by low values of σ for incipient damage. For example, a carefully designed gate slot, with an offset and rounded downstream corner, may have a damage σ as low as 0.2. Unfortunately, the lowest value of σ a designer can use may be fixed by unintentional surface imperfections in concrete, the need for small abrupt expansions in flow passages, or the likelihood that vortices will be generated by obstructions such as partially open sluice gates. To be realistic, one may have to expect boundary geometry that will cause cavitation damage, if σ drops below about 1.2.

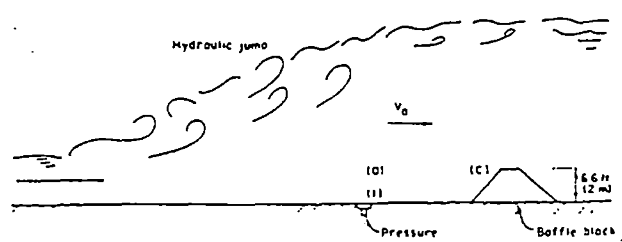
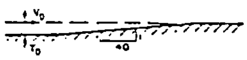
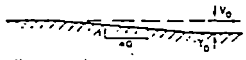
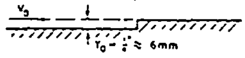
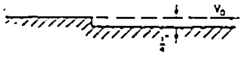


Fig. 5.1 — Baffle block downstream from a low spillway.

Structure or Irregularity	σ	References
Tunnel inlet	1.5	Tullis 1981
Sudden expansion in tunnel	1.0* 0.19	Russell and Ball 1967 Rouse and Jezdinsky 1966
Baffle blocks	1.4 & 2.3	Galperin et al. 1977
Gates and gate slots	0.2 to 3.0	Galperin et al. 1977 Ball 1959 Wagner 1967
Abraded concrete 3/4 in. max. depth of roughness	0.6	Ball 1976
	0.2	Ball 1976 Arndt 1977 Falvey 1982
	0.2	
	1.6	
	1.0	

*Unusual definition of σ

Fig. 5.2 — Values of σ at beginning of cavitation damage

A third choice, often inevitable, is to expect cavities to form at predetermined locations. In this case, the designer may: a) supply air to the flow, or b) use damage-resistant materials such as stainless steel, fibrous concrete, or polymer concrete systems.

Using damage-resistant materials will not eliminate damage, but may extend the useful life of a surface. This alternative is particularly attractive, for example, for constructing or repairing outlet works that will be used infrequently or abandoned after their purpose has been served.

In any case, values of σ at which cavitation erosion begins are needed for all sorts of boundary geometries. Sometimes critical values of σ may be estimated by theory, but they usually come from model or prototype tests.

5.2 — Cavitation indexes for damage and construction tolerances

Fig. 5.2 lists a few values of σ at which cavitation begins and the references from which these values came. A designer should not use these numbers without studying the references. Some reasons for this are:

a. The exact geometry and test circumstances must be understood.

b. Authors use different locations for determining the reference parameters of Eq. (2-1). However, the general form of Eq. (2-1) is accepted by practitioners in the field.

c. Similitude in the model is difficult to achieve.

Many of the essential details involved in the original references are explained in Hamilton (1983 and 1984) which deals with the examples in Fig. 5.2.

The values of σ listed in Fig. 5.2 show the importance of good formwork and concrete finishing. For example, a ¼-in. (6-mm) offset into the flow which could be caused by mismatched forms has a σ of 1.6, whereas a 1:40 chamfer has a σ only one-eighth large. By the definition of σ, the allowable velocity the chamfer would be $\sqrt{8}$ times the allowable velo past the offset if $p_0 - p_v$ were the same in both ca Thus, on a spillway or chute where $p_0 - p_v$ migh 17.4 psi (120 kPa), damage would begin behind offset when the local velocity reached 40 ft/sec (12 sec), but the flow past the chamfer would cause trouble until the velocity reached about 113 ft/sec m/sec).

When forms are required, as on walls, ceilings, steep slopes, expert workmen may produce a ne smooth and only slightly wavy surface for which σ be as low as 0.4. Using the preceding $p_0 - p_v$ giv damage velocity of 80 ft/sec (24 m/sec). A σ valu 0.2, on which the 113 ft/sec (35 m/sec) is based, be achieved on plane, nearly horizontal surfaces by ing a stiff screed controlled by steel wheels running rails and hand floating and troweling.

Construction tolerances should be included in all sign and repair specifications. These establish per sible variation in dimension and location giving t the designer and the contractor parameters wi which the work is to be performed. ACI 117 prov guidance in establishing practical tolerances. sometimes necessary that the specifications for c crete surfaces in high-velocity flow areas be even n demanding. However, achieving more restrictive to ances for hydraulic surfaces than those recommer by ACI 117 can become very costly or even impra cal. The final specification requirements require ju ment on the part of the designer.

Joints can cause problems in meeting tolerances, e with the best workmanship. Some engineers prefe saw and break out areas where small offsets oc rather than to grind the offsets that are outside specification. The trough or hole is then patched hand finished in an effort to produce a surface n resistant to erosion than a ground surface would be some cases grinding to achieve alignment and smo ness is adequate. However, to help prevent the oc rence of aggregate popouts, a general rule of thum to limit the depth of grinding to one-half the maxin diameter of the coarse aggregate. Ground surfaces also be protected by applying a low-viscosity, pene ing phenol epoxy-resin sealer (Borden et al. 1971).

The difficulty of achieving a near-perfect surface the doubt that such a surface would remain smo during years of use have led to designs that permit introduction of air into the water to cushion the lapse of cavities when low pressures and high veloc prevail.

5.3 — Using aeration to control damage

Laboratory and field tests have shown that sur irregularities will not cause cavitation damage if the water ratio in the layers of water near the solid bou ary is about 8 percent by volume. The air in the w should be distributed rather uniformly in small b bl

When calculations show that flow without aeration is likely to cause damage, or when damage to a structure has occurred and aeration appears to be a remedy, the problem is dual: a) the air must be introduced into the flowing water and b) a portion of that air must remain near the flow/concrete boundary where it will be useful.

The migration of air bubbles involves two principles: a) bubbles in water move in a direction of decreasing water pressure, and b) turbulence disperses bubbles from regions of high air concentration toward regions of low concentration.

Careful attention must be given to the motion of bubbles due to pressure gradients. A flow of water surrounded by atmospheric pressure is called a free jet. In a free jet, there are no gradients except possibly weak local ones generated by residual turbulence, and the bubbles move with the water. There is no buoyant force. On a vertical curve that is convex, the bubble motion may have a component toward the bottom. In a flip bucket, which is concave, the bottom pressure is large and the bubbles move rapidly toward the free surface.

When aeration is required, air usually must be introduced at the bottom of the flow. These bubbles gradually move away from the floor in spite of the tendency for turbulent dispersion to hold them down. At the point where insufficient air is in the flow to protect the concrete from damage, a subsequent source of bottom air must be provided.

Aeration data measured on Bratsk Dam in the U.S.S.R., which has a spillway about 295 ft (90 m) high and an aeration device, has been discussed by Semenkov and Lentyaev (1973) (see Table 5.1). Downstream from the aeration ramp, measurements showed that the air-water ratio in a 6-in. (150-mm) layer next to the concrete declined from 85 to 35 percent as the mixture flowed down the spillway a distance of 174 ft (53 m). If one assumes an exponential type of decay, the loss per foot was a little less than 2 percent of the local air-water ratio.

It is usually not feasible to supply air to flowing water by pumping or compressing the air because the volumes involved are too large. Instead, the flow is pro-

Table 5.1 — Examples of use of air to prevent cavitation damage

Structure or description	References
Palisades Dam outlet sluices (USA)	Beichley and King 1975
Yellowtail Dam spillway tunnel (USA)	Borden et al. 1971, Colgate 1971
Glen Canyon Dam spillway tunnel (USA)	Burgi, Moyes, and Gamt 1984
Ust-Ilim Dam spillway (USSR)	Oskolkov and Semenkov 1979
Bratsk Dam spillway (USSR)	Semenkov and Lentyaev 1973
Foz do Areia spillway (Brazil)	Pinto et al. 1982
General	Galperin et al. 1977
Comprehensive	Hamilton 1983 and 1984, Quintela 1980

jected from a ramp or step as a free jet, and the w introduces air at the air-water interfaces. Then the bulence within the jet disperses the air entrained at interfaces into the main body of the jet. Fig. 5.3 sh typical aeration ramps for introducting air into the f (Wei and DeFazio 1982).

To judge whether sufficient air will remain adja to the floor of a spillway, the amount of air that a bulent jet will entrain must be estimated. On dim sional grounds, the following equation for entrainm by the lower surface has been proposed (Hamilton 1 and 1984)

$$q_a = cV\ell \qquad ($$

in which q_a = volume rate of air entrainment per u width of jet

c = coefficient

V = average jet velocity at midpoint of t jectory

ℓ = length of air space between the jet a the spillway floor.

Model and prototype measurements indicate that value of the coefficient c lies between 0.01 and 0. depending upon velocity and upstream roughness.

The length of cavity ℓ (Fig. 5.3) is difficult to m sure in prototypes and large models. Instead, the up

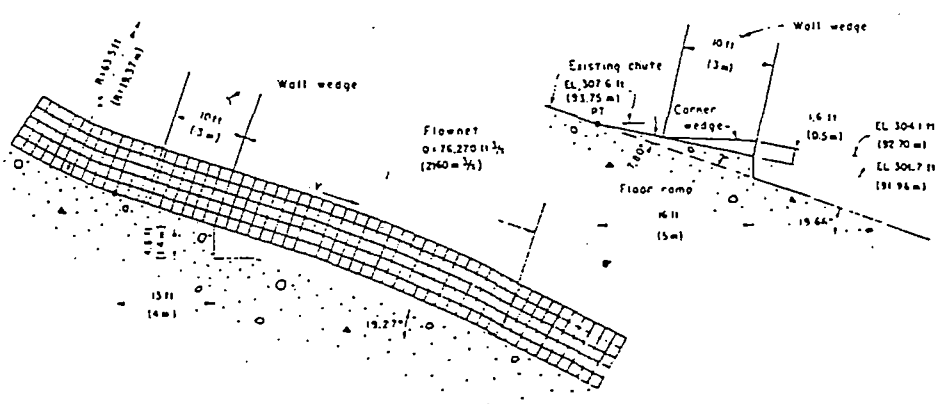
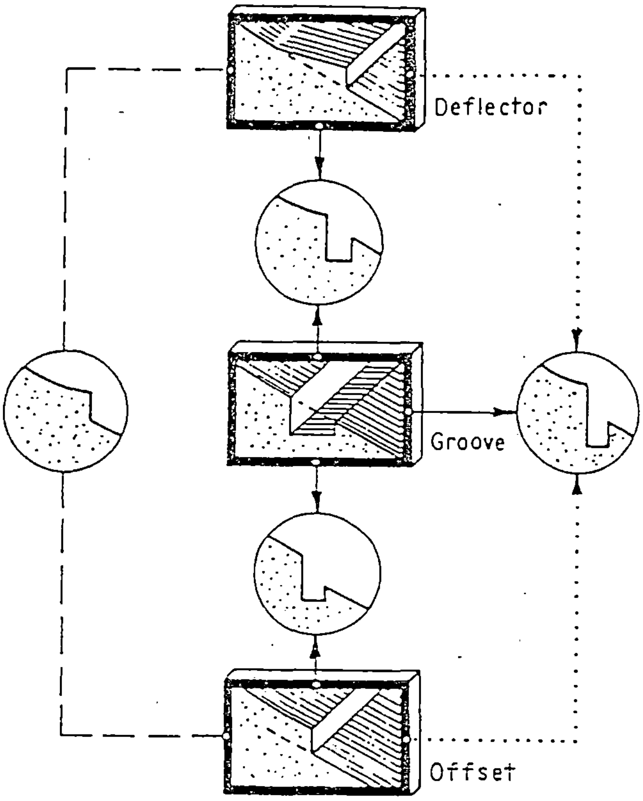
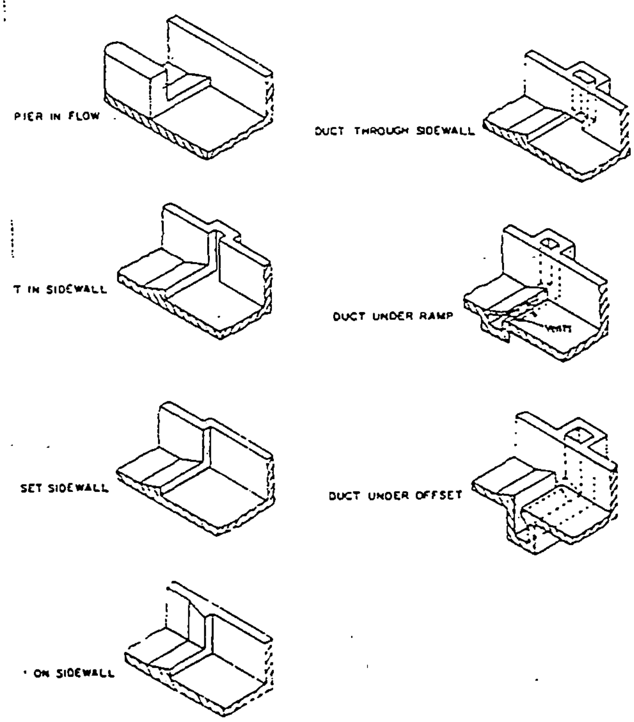


Fig. 5.3 — Aeration ramps at King Talal Spillway



:ig. 5.4 — *Types of aerators (from Vischer, Volkart, 'nd Siegenthaler 1982)*



. 5.5 — *Air supply to aerators (from Falvey, in 'lication)*

lower profiles of the nappe can be estimated from -dimensional irrotational flow theory. One method ɔ use a finite element technique for calculating pe trajectories.

As indicated above, ramps and down-steps are ւ to induce the flow in a spillway or tunnel to spring from the floor. A ramp is a wedge anchored to or tegral with the floor and usually spans the tunnel spillway bay. Ramps vary in length from 3 to 9 ft (' 3 m). Wall and corner wedges and wall offsets a\ from the flow also are used to cause the water to le the sides of a conduit. The objective is to provid sudden expansion of the solid boundaries. Such vices, often referred to as aerators, are visually picted in Fig. 5.4 and 5.5. (See also Ball 1959, DeFa and Wei 1983, and Russell and Ball 1967.)

Air is allowed to flow into a cavity beside or unde jet by providing passages as simple as the layout of project will permit. Sometimes the required rates airflow are enormous. For example, a cavity und neath a spillway nappe 49 ft (15 m) wide could entr 5160 ft^3/sec (146 m^3/sec) of air. A single passageway least 6.6 ft (2.0 m) in diameter would be needed to sı ply this amount.

Although offsets, slots, and ramps in conduits c introduce air into high-velocity flow to effectively cc trol cavitation, if improperly designed they can acc tuate the cavitation problem. For this reason, it is a visable to conduct physical hydraulic model studies insure the adequacy of a proposed aeration device.

5.4 — Fatigue caused by vibration

In concrete, flexural fatigue is normally thought of terms of beams bending under repeated relatively hi amplitudes and low-frequency loads. A mass of cc crete at the surface of an outlet or spillway ordinari does not bend, but it does vibrate. In this case, the d formation is three-dimensional with low amplitude aı high frequency. For instance, at McNary Dam the ' bration was measured as 0.00002 in. (0.00051 mm) aı 150 cycles per second (cps) for the transverse directio Unfortunately, there are no reported studies of co crete fatigue caused by vibration.

A vibration test for concrete and epoxy/polym materials is needed. Data from such a test would l useful for evaluating various construction and repa materials. A standard test has been developed for sma samples of homogeneous materials which vibrates tl sample at 20,000 cps and 0.002 in. (0.051 mm) amp. tude while it is submerged in the fluid. Stilling bas. floors, walls, and outlets are essentially full-scale tes of the same type.

5.5 — Materials

Although proper material selection can increase tł cavitation resistance of concrete, the only totally effe tive solution is to reduce or eliminate the factors th trigger cavitation, because even the strongest materia cannot withstand the forces of cavitation indefinitel The difficulty is that in the repair of damaged struc tures, the reduction or elimination of cavitation may b very difficult and costly. The next best solution is t replace the damaged concrete with more erosion-resis tant materials.

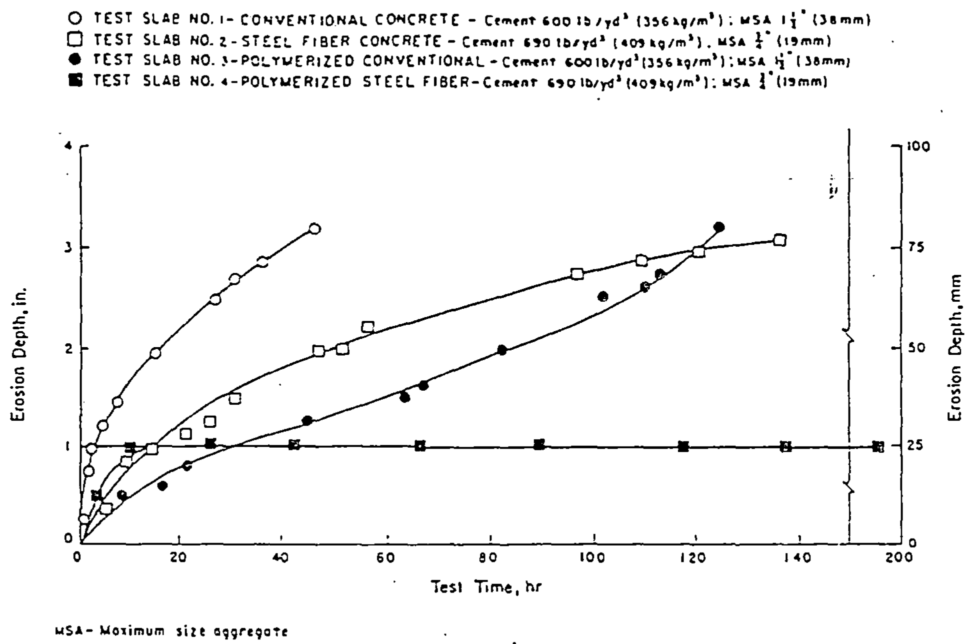


Fig. 5.6 — Erosion depth versus time, Tarbela Dam concrete mixtures

In areas of new design where cavitation is expected to occur, designers may include the higher quality materials during the initial construction or include provisions for subsequent repairs in service. For example, in many installations, stainless steel liners are installed on the concrete perimeter downstream of slide gates to resist the damaging effects of cavitation. These liners, although quite durable, may pit and eventually have to be replaced.

The cavitation resistance of concrete where abrasion is not a factor can be increased by using a properly designed low water-cement ratio, high-strength concrete. The use of aggregate no larger than 1½ in. (38 mm) nominal maximum size is recommended, and the use of water-reducing admixtures and chilled concrete has proven beneficial. Hard, dense aggregate and good bond between aggregate and mortar are essential to achieving increased cavitation resistance.

Cavitation-damaged areas have been successfully repaired using steel fiber reinforced concrete (ICOLD 1982). This material exhibits good impact resistance necessary to resist the many tiny point loads and appears to assist in arresting cracking and disintegration of the concrete matrix. The use of polymers as a matrix binder or a surface binder has also been found to improve substantially the cavitation resistance of both conventional and fibrous concrete (Schrader 1978 and 1983b).

Some coatings, such as neoprene or polyurethane, have effectively reduced cavitation damage to concrete, but since near-perfect adhesion to the concrete is mandatory, the use of such coatings is not common. Once there is a tear or a chip in the coating, the entire coating is soon peeled off.

5.6 — Materials testing

Because of the massive size of most hydraulic structures, full-scale prototype testing is usually not possible. Model testing can identify many potential problem areas, but determining the ultimate effect of hydraulic forces on the structure requires some judgment. In some cases, it is desirable to evaluate a material after it has been subjected for a reasonable period of time to flows of a magnitude approaching that expected during operation of the facility.

The U.S. Army Corps of Engineers has been evaluating erosion resistance of materials at the Detroit Dam (Oregon) High Head Erosion test flume (Houghton, Borge, and Paxton 1978). Erosion testing at the facility consists of preparing test slabs 21 in. (530 mm) wide by 10 ft (3 m) long using the desired material, coating, or overlay. High-velocity water is passed over the slabs for various durations, and the performance of the material is then evaluated. Cavitation erosion resistance is studied by embedding small obstacles in the test slabs which protrude into the flow (Fig. 2.5).

Materials and coating systems tested at the Detroit Dam facility include various concrete mixes, fibrous concrete, roller-compacted concrete, polymer-impregnated concrete, polymer-impregnated fibrous concrete, and several concrete coatings (Houghton, Borge, and Paxton 1978). Fig. 5.6 shows the performance of several of these materials subjected to flows with velocities of 120 ft/sec (37 m/sec).

5.7 — Construction practices

Construction practices are of paramount importance when hydraulic surfaces may be exposed to high-velocity flow, particularly if aeration devices are not incorporated in design. Such surfaces must be as smooth as can be practically obtained (Schrader 1983b). Surface imperfections and deficiencies have been known to cause cavitation damage at flow velocities as low as 26 ft/sec (8 m/sec). Offsets no greater than ⅛ in. (3 mm) in height have been known to cause cavitation damage at flow velocities as low as 82 ft/sec (25 m/sec). Patching repairs improperly made at the time of construction have been known to fail under the stress of water flow or for other reasons, thereby providing the surface imperfections which triggered cavitation damage to the concrete farther downstream. This phenomenon occurred in the high head spillway tunnel at Yellowtail Dam, Montana, ultimately resulting in major cavitation and structural damage to the concrete lining (Borden et al. 1971; Colgate 1971). Accordingly, good construction practices as recommended in ACI 117, ACI 302.1R, ACI 304, ACI 308, ACI 309, and ACI 347 should be maintained both for new construction and repair. Formed and unformed surfaces should be carefully checked during each construction operation to confirm that they are within specific tolerances.

If the potential for cavitation damage exists, care should be taken in placing the reinforcement. The bars closest to the surface should be placed parallel to the direction of flow so as to offer the least resistance to flow in the event that erosion reaches the depth of the reinforcement. Extensive damage has been experienced where the reinforcement near the surface is normal to the direction of flow.

Where possible, transverse joints in concrete conduits or chutes should be minimized. These joints are generally in a location where the greatest problem exists in maintaining a continuously smooth hydraulic surface. One construction technique which has proven satisfactory in placement of reasonably smooth hydraulic surfaces is the traveling slipform screed. This technique can be applied to tunnel inverts and to spillway chute slabs. Information on the slipform screed can be found in Hurd (1979).

Proper curing of these surfaces is essential, since the development of surface hardness improves cavitation resistance.

CHAPTER 6 — CONTROL OF ABRASION EROSION

6.1 — Hydraulic considerations

Under appropriate flow conditions and transport of debris, all of the construction materials currently being used in hydraulic structures are to some degree susceptible to abrasion. While improvements in materials should reduce the rate of damage, these alone will not solve the problem. Until the adverse hydraulic conditions which can cause abrasion erosion damage are minimized or eliminated, it is extremely difficult for any of the construction materials currently being used to perform in the desired manner. Prior to construction or repair of major structures, hydraulic model studies of the structure should be conducted to identify potential causes of erosion damage and evaluate the effectiveness of various modifications in eliminating the undesirable hydraulic conditions. If the model test sults indicate it is impractical to eliminate the undes able hydraulic conditions, provisions should be made design to minimize future damage. For example, go design practices should consider the following m sures in the construction or repair of stilling basins:

a. Include provisions such as debris traps or low vision walls to minimize circulation of debris.

b. Avoid use of baffles which are connected to st ing basin walls. Alternatively, considering their susc tibility to erosion, avoid use of appurtenances such chute blocks and baffles altogether when the desi makes this possible.

c. Use model tests for design and detailing of the t minus of the stilling basin and the exit channel, so as maximize flushing of the stilling basin and to minim chances of debris from the exit channel entering t basin.

Maintain balanced flows into the basins of existi structures, using all gates, to avoid discharge con tions where flow separation and eddy action are pre lent. Substantial discharges that can provide a go hydraulic dump without creating eddy action should released periodically in an attempt to flush debris fr the stilling basin. Guidance as to discharge and tail ter relations required for flushing should be develop through model or prototype tests, or both. Periodic spections should be required to determine the preser of debris in the stilling basin and the extent of er sion. If the debris cannot be removed by flushing c erations, water releases should be shut down and t basin cleaned by other means.

6.2 — Material evaluation

Materials should be tested and evaluated prior being used in hydraulic structures subjected to abrasi erosion damage. A variety of test methods includi rubbing types of apparatus; dressing wheel; rolling st balls under pressure (ASTM C 779); sandblasti (ASTM C 418); and modified Los Angeles rattl (ASTM C 131 and C 535) have been used to determi abrasion erosion resistance of concrete surfaces. The tests, designed to simulate heavy foot or wheeled traf on concrete surfaces, are not intended to model ab sion by waterborne particles.

The U.S. Army Corps of Engineers' test CRD-C 6 80, "Test Method for Abrasion-Erosion Resistance Concrete (Underwater Method)," is a better model the abrasive action of waterborne particles on a h draulic structure. This test procedure subjects concr specimens to abrasion erosion under the action of st grinding balls. The steel grinding balls are propelled water in the test chamber. The water is in turn prop led by a submerged mixer paddle. Water velocity on t surface of the specimen is approximately 6 ft/sec (2 r sec). Test specimens are periodically removed from t apparatus to determine the amount of abrasion erosi damage. The damage is quantified and the lost ma rial is reported as a percentage of original mass. T development of the test procedure and data from a large variety of tests of various concrete mixtures have been described by Liu (1980).

6.3 — Materials

A number of materials and techniques have been used in the construction and repair of structures subjected to abrasion erosion damage, with varying degrees of success. The degree of success is inversely proportional to the degree of exposure to those conditions conducive to erosion damage (McDonald 1980). No single material has shown consistently superior performance when compared to others. Improvements in materials are expected to reduce the rate of concrete damage due to abrasion erosion. The following factors should be considered when selecting abrasion-resistant materials.

Abrasion-resistant concrete should include the maximum amount of the hardest available coarse aggregate and the lowest practical water-cement ratio. The abrasion-erosion resistance of concrete containing chert aggregate has been shown to be approximately twice that of concrete containing limestone (Fig. 6.1). Given a good, hard aggregate, any practice that produces a stronger paste structure will increase abrasion-erosion resistance. In some cases where hard aggregate was not available, high-range water-reducing admixtures and silica fume have been used to develop very strong concrete — that is, concrete with a compressive strength of about 15,000 psi (103 MPa) — and to overcome problems with unsatisfactory aggregate (Holland 1983). Apparently, at these high compressive strengths, the hardened cement paste assumes a greater role in resisting abrasion-erosion damage and the aggregate quality becomes correspondingly less important.

Concrete, when produced with shrinkage-compensating cement, and when properly proportioned and cured, has an abrasion resistance from 30 to 40 percent higher than portland cement concrete of comparable mixture proportions, according to Committee 223 (1970) and Klieger and Greening (1969).

While the addition of steel fibers may be expected to increase the impact resistance of concrete, some data show that fiber-reinforced concrete is less resistant to abrasion erosion than conventional concrete (Liu and McDonald 1981). This is attributed primarily to the fact that fiber-reinforced concrete generally has less coarse aggregate per unit volume of concrete than does comparable conventional concrete.

The abrasion-erosion resistance of vacuum-treated concrete, polymer concrete, polymer-impregnated concrete, and polymer-portland cement concrete is significantly superior to that of comparable conventional concrete. This is attributed to a stronger cement matrix. The increased costs associated with materials, production, and placing of these and any other special concretes in comparison with conventional concrete should be considered during the evaluation process.

Several types of surface coatings have exhibited good abrasion-erosion resistance in laboratory tests. These

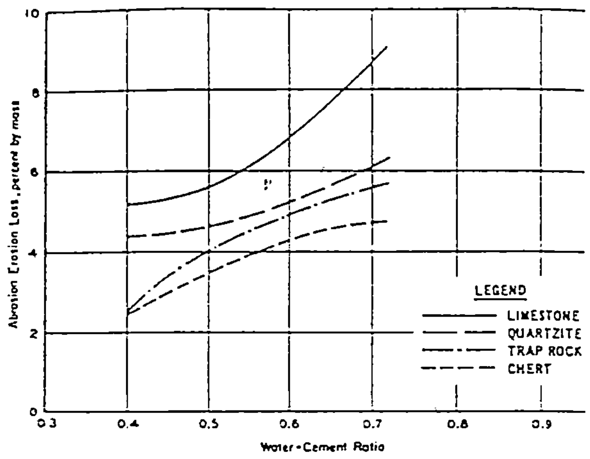


Fig. 6.1 — Relationships between water-cement and abrasion-erosion loss

include polyurethanes, epoxy-resin mortar, furan-mortar, acrylic mortar, and iron-aggregate topp Problems in field application of surface coatings been reported (McDonald 1980). These have been primarily to improper surface preparation or the incompatibility between coatings and concrete. N recently, formulations have been developed which coefficients of thermal expansion more similar to of the concrete substrate.

CHAPTER 7 — CONTROL OF EROSION B' CHEMICAL ATTACK

7.1 — General

Retention of waste water in anaerobic condit may result in hydrogen sulfide (H_2S) generation. Th turn may have detrimental effects on concrete in tially full sanitary sewers which convey waste wat low velocities. The process of sulfide generation sanitary sewer when insufficient dissolved oxyge present in the waste water has been discussed and i trated by an ASCE-WPCF Joint Task Force (19 This original work was performed by Pomeroy (19 The practicing engineer involved with projects of nature would be wise to review also the excellent ommendations set forth in the ACPA *Concrete i Handbook.*

7.2 — Control of erosion by mineral-free watei

The mild acid attack possible with pure water ra develops into deterioration that can cause severe st tural damage. Generally, the mineral-free water leach mortar on surfaces exposed to this water. can be seen on exposed surfaces and at joints cracks in concrete sections. As the surface mort leached from the concrete, more coarse aggregate is posed, which naturally decreases the amount of mo exposed. With less mortar exposed, less leaching curs, and hence major structural problems do not i ally result. The gradual erosion of the leached mo can be minimized by use of special cements, additio pozzolan to mixes, or use of a variety of protective coatings and sealants applied to concrete surfaces (Tuthill 1966).

7.3 — Control of erosion from bacterial action

Concrete conduits have served in sewer systems for many years without serious damage where the systems were properly designed and operated. The minimum adequate velocity of flow in the sewer for the strength and temperature of the sewage is usually 2 ft/sec (0.6 m/sec). Providing this velocity without excessive turbulence and providing proper ventilation of the sewer will generally prevent erosion by bacterial action. Turbulence is to be avoided because it is an H_2S releasing mechanism. Where conditions are such that generation of H_2S cannot be totally eliminated by the design of the system, then other means may be applied, such as: 1) using hydrogen peroxide or chlorine compounds to convert the H_2S (WPCF 1979)

(a) $H_2O_2 + H_2S \rightarrow 2H_2O + S$

(b)(1) $Cl_2 + H_2O \rightarrow HOCl + H^+ + Cl^-$

(2) $HOCl + H_2S \rightarrow S + HCl + H_2O$ (low pH)

(3) $S^{2-} + 4Cl_2 + 8OH^- \rightarrow SO_4^{2-} + 8Cl + 4H_2O$ (high pH)

2) introducing compressed air to keep sewage fresh and thereby prevent the development of the anaerobic environment; 3) using an acid-resistant pipe such as vitrified clay or polyvinyl chloride (PVC) pipe; 4) using acid-resisting liners on the crown of sewers; and/or 5) increasing the concrete section to allow a sacrificial thickness based on predicted erosion rates. Graphical methods have been published for determining sulfide buildup in sanitary sewers, using the Pomeroy-Parkhurst equations (Kienow et al. 1982).

Parker (1951) lists the following remedial measures for the control of H_2S attack in concrete sewers:

I. Reduction-potential-generation
- inflow reduction
- partial purification
- chemical dosage to raise oxidation (but addition of nitrates is impracticable)
- aeration

Table 7.1 — Recommended cement types to use in concrete when mixing water contains sulfates

mg/l sulfate (as SO_4) in water	Cement type
0-150	Any type
150-1500	Type II, IP
1500-10,000	Type V, or Type I or II with a pozzolan which has been shown by test to provide comparable sulfate resistance when used in concrete, or Type K shrinkage-compensating
10,000 or more	Type V plus an approved pozzolan which has been determined by tests to improve sulfate resistance when used in concrete along with Type V

- chlorination
- removal of slimes and silts
- velocity increase

II. Emissions
- turbulence reduction
- treatment with heavy metal salts (Cu, Fe, Z
- treatment with alkalies
- full flow in sewer

III. H_2S fixation on concrete
- ventilation
- periodic wetting
- use of resistant concrete
- ammoniation
- use of protective coatings

The engineer faced with reducing bacterial acti should be aware that a) chlorination may, under ce tain circumstances, be illegal because it can produ trihalomethane, a known carcinogen; and b) it m also be illegal to add lead salts (which usually are t only cost-effective choice) or other heavy metal salts waste water.

Further information on remedial measures for sa tary sewer systems is available in U.S. Environmen Protection Agency publication EPA/625/1-85/C (1985).

7.4 — Control of erosion by miscellaneous chemical causes

7.4.1 *Acid environments* — No portland ceme concrete, regardless of its other ingredients, will wi stand attack from water of high acid concentratic Where strong acid corrosion is indicated, other cc struction materials or an appropriate surface cover or treatment should be used. This may include ap cations of sulfur-concrete toppings, epoxy coatin polymer impregnation, linseed-oil treatments, or oth processes, each of which affects acid resistance diff ently. Replacement of a portion of the portland cem by a suitable amount of pozzolan selected for t property can improve the resistance of concrete to w acid attack. Also, limestone or dolomite aggrega have been found to be beneficial in extending the life structures exposed to acid attack (Biczók 1967).

Deterioration similar to that which occurs in crown of sewers has also occurred above water leve tunnels which drain lakes, the waters of which cont sulfur and other materials that are susceptible to formation of hydrogen sulfide by bacterial action.

7.4.2 *Alkali-aggregate reaction and chloride adm tures* — Deterioration of concrete caused by alkali- gregate reaction and by chloride admixtures in the c crete mixture is not included in this discussion. Tut (1966) and ACI 201.2R provide information on th topics.

7.4.3 *Soils and ground waters* — Sulfates of sodi magnesium, and calcium frequently encountered in "alkali" soils and ground waters of the western Un States attack concrete aggressively. ACI 201.2R cusses this in detail. Use of Type V sulfate-resisting ment, low in C_3A, is recommended whenever the fate in the water is within the ranges shown for its use in Table 7.1. Types I and II cements are usually relatively high in C_3A. Concrete mixtures using Types I or II portland cement can be improved with respect to sulfate resistance by replacing approximately 30 percent of the portland cement with a suitable natural pozzolan or fly ash. Rich mixes are more resistant to sulfate attack than lean ones. The use of shrinkage-compensating cements, made with Type II or Type V portland cement clinker and adequately sulfated, produces concrete having sulfate resistance equal to or greater than portland cement made of the same type clinker (Mehta and Polivka 1975). Table 7.1 lists the recommended cement types for corresponding sulfate contents.

PART 3 — MAINTENANCE AND REPAIR OF EROSION

Chapter 8 — Periodic inspections and corrective action

8.1 — General

The regular, periodic inspection of completed and operating hydraulic structures is extremely important. The observance of any erosion of concrete should be included in these inspections. The frequency of inspections is usually a function of use and evidence of distress. The inspections provide a means of routinely examining structural features as well as observing and discussing problems needing remedial action. ACI 201.1R, ACI 207.3R, and U.S. Department of the Army publication EM-1110-2-2002 (1979) provide detailed instructions for conducting extensive investigations.

8.2 — Inspection program

The inspection program must be tailored to the specific type of structure. The designers should provide input to the program and identify items of primary and secondary importance. The actual inspection team should be composed of qualified technical personnel who know what to look for and can relate in common terminology. The size of the team is generally dependent on the number of technical disciplines required. The program should be established and monitored by an engineer who is experienced in design, construction, and operation of the project.

8.3 — Inspection procedures

Prior to the on-site inspection, the team should thoroughly evaluate all available records, reports, and other documentation on the condition of the structure and maintenance and repair, and become familiar with previous recommendations. Some of the more important observations to make during an examination of hydraulic facilities are:

a. Identifying structural cracking, spalling, and displacements within the water passage
b. Identifying surface irregularities
   1. Offset into or away from flow
   2. Abrupt curvature away from flow
   3. Abrupt slope away from flow
   4. Void or transverse groove
   5. Roughened or damaged surfaces which gi evidence of cavitation or abrasion erosion
   6. Structural imperfections and calcite deposi
   7. Cracking, spalling, and rust stains from inforcement
c. Inspecting gate slots, sills, and seals, includi identification of offsets into the flow
d. Locating concrete erosion adjacent to embedd steel frames and steel liners and in downstre water passages
e. Finding vibration of gates and valves during op ation
f. Observing defective welded connections and t pitting and/or cavitation of steel items
g. Observing equipment operation and maintenanc
h. Making surveys and taking cross sections to det mine the extent of damage
i. Investigating the condition of concrete by nond tructive methods or by core drilling and samplir if distressed conditions warrant
j. Noting the nature and extent of debris in wa passages

All conditions observed and their exact locatio should be accurately recorded by the inspection te for future reference. High-quality photographs of c ficiencies are extremely beneficial and provide a p manent record which assists in identifying slow pr gressive failures. A report should be written for ea inspection to record the condition of the project and justify funding for repairs.

8.4 — Reporting and evaluation

The inspection report may vary from a formal pu lication to a trip report or letter report. The repc should include the standard items: who, why, wha where, and when. A pre-established outline is usually value. An inspection checklist of deficiencies and su sequent corrective actions should be established frc prior inspections. Any special items of interest may shown in sketches or photos. The report should a dress existing and potential problems, and it shou categorize the deficiencies relative to the urgency corrective action and identify the extent of damag probable cause of damage, and probable extent damage if immediate repairs are not made. It is e tremely important that the owner or agency distribu the report in accordance with any applicable U.S. fe eral or state safety regulations.

When the inspection report indicates that remed action is required, the next step may be either a supp mental investigation or the actual corrective actio Deficiencies noted in the inspection should be eval ated and categorized as to minor, major, or potentia catastrophic. The scope of work should be defined early as possible in order to establish reliable budg estimates. Design for proper repair schemes sometim requires model tests, redesign of portions of the stru ture, and materials investigations. Each of these iter requires funding through the owner's program of operations. The more details identified in the scope of work, the more accurate the cost estimate. Wherever possible, it is important to correct the probable cause, so that the repairs will not have to be repeated in the near future.

CHAPTER 9 — REPAIR METHODS AND MATERIALS

9.1 — Design considerations

9.1.1 *General* — It is desirable to eliminate the cause of the erosion whenever possible; however, since this is not always possible, a variety of materials and material combinations is used for the repair of concrete. Some materials are better suited for certain repairs, and judgment should be exercised in the selection of the proper material. Consideration also should be given to the time available to make repairs, access points, logistics in material supply, ventilation, nature of the work, available equipment, and skill and experience of the local labor force.

Detailed descriptions of repair considerations and procedure may be found in the U.S. Bureau of Reclamation's *Concrete Manual* (1981).

9.1.2 *Consideration of materials* — A major factor which is critical to the success of a repair is the relative volume change between the repair material and the concrete substratum. Many materials change volume as they initially set or gel, almost all change volume with changes in moisture content, and all change volume with changes in temperature. If a repair material decreases sufficiently in volume relative to the concrete, it will develop cracks perpendicular to the interface, generally at a spacing related to the repair depth. Shear and tensile stresses also will develop at the interface with a maximum magnitude at the tip of each crack, and the stresses will cycle with each temperature and moisture cycle. ASTM C 884 evaluates a specific class of materials with respect to temperature change. Similar tests should be applied to all repair materials.

Since differential volume change imposes stresses at an interface between a repair material and the concrete, suitable preparation of that interface is essential to the success of the repair. Sound concrete may not be able to resist stresses imposed by a high volume change repair material, whereas it may resist those imposed by a low volume change material. ACI 503R has recommended that the interface between concrete and epoxy patches exhibit an absolute minimum tensile strength, by a specific test method, of 100 psi (0.69 MPa).

Normal portland cement concrete is generally the least expensive replacement material and will most nearly match the characteristics of the in-place concrete with regard to temperature change. Normal concrete will almost certainly be subject to an initial shrinkage relative to the original concrete and possibly thermal stresses from heat of hydration if the depth of replacement is sufficient to develop a significant temperature gradient within the repair.

The best way to minimize plastic and drying sh age is to minimize water content in the replac concrete. Thus, stiff mixtures, with or without i corporation of polymers or copolymers as a re ment for part of the mix water, may be consic Stiff mixtures may require careful use of bo agents and be more difficult to place and consol It also may be difficult to consolidate stiff mi around reinforcing steel. The use of polymers ca prove the useability of the concrete, but also sul tially increases material costs, may present addi handling hazards, and may require special constr techniques.

9.2 — Methods and materials

9.2.1 *Steel plating* — Installing stainless steel plates on concrete surfaces subject to high-vel flows has been a generally successful method of tecting the concrete against cavitation erosion. gate's (1977) studies show stainless steel to be a four times more resistant to cavitation damage tha dinary concrete. The currently preferred stainless terial is ASTM A 167, S30403 (formerly SS304L), the standpoint of excellent corrosion and cavitatic sistance, and weldability. The steel plates must b curely anchored in place and be sufficiently sti minimize the effects of vibration. Vibration of the plate can lead to fracturing and eventual failure o underlying concrete or failure of the anchors. U tunately, the steel plating may hide early signs of crete distress.

This repair method, like many others, treats onl symptom of erosion and eventually, if the cavitati not reduced or eliminated, the steel itself may bec damaged by pitting.

9.2.2 *Dry-packed concrete* — Use of dry-pa concrete is generally limited to applications where material can be tamped into cavities which have a d at least as great as their width. These limited app tions are necessary because the material is friable compacted in place by tamping or ramming. The water content of the dry-pack, combined with the sity obtained by the compaction process, gives a p that will experience very little drying shrinkage and have expansion properties similar to the parent crete.

Dry-pack should consist of one part cement to parts masonry sand (passing No. 16 screen). La and other special admixtures can be used in the mi when bonding or another special characteristic is sired. The consistency of the dry-pack mortar shoul such that when balled in the hand, the hand is m but not dirty. White cement can be blended with cement if appearance is important. The completed w should be moist-cured, just as any concrete.

Dry-packed concrete repairs, as is true of all rep require care on the part of both designer and const tor to insure that the final product meets the inter design. Properly made, dry-packed concrete rep have proven to be very satisfactory.

"Damp-pack," a similar material discussed in U.S. Army Corps of Engineers Technical Report MRDL 2-74 (1974) and the *ACI Manual of Concrete Inspection* (1981), can be sprayed onto existing concrete for repair of peeled areas and other shallow defects.

9.2.3 *Fiber reinforced concrete (FRC)* — Conventional concrete typically performs poorly where the following material properties are important to the life of the structure or its performance: fatigue strength, cavitation and abrasion-erosion resistance, impact strength, flexural strength and strain capacity, post-cracking load-carrying capability, and high shear strength. FRC utilizes randomly oriented discrete fiber reinforcement in the mixture and offers a practical way of obtaining these properties for most applications. ICOLD Bulletin 40 (1982) describes its use in dams. FRC has been successfully used in some erosion situations. There are examples where FRC repairs have been made that demonstrate that fibrous concrete is resistant to the combined effects of cavitation and abrasion erosion by large rock and debris carried at high velocity. On the other hand, laboratory abrasion-erosion tests under conditions of low velocity carrying small-size particles have shown that the addition of fibers may not be beneficial, and in fact may be detrimental (Liu and McDonald 1981). ACI 544.1R and ACI publication SP-81 provide additional information regarding the use of FRC.

9.2.4 *Epoxy resins* — Resins are natural or synthetic, solid or semisolid organic materials of high molecular weight. Epoxies are one type of resin. These materials are typically used in preparation of special coatings or adhesives or as binders in epoxy-resin mortars and concretes. Several varieties of resin systems are routinely used for the repair of concrete structures. ACI 503R describes the properties, uses, preparations, mixtures, application, and handling requirements for epoxy resin systems.

The most common use of epoxy compounds is in bonding adhesives. Epoxies will bond to most building materials, with the possible exception of some plastics. Typical applications include the bonding of fresh concrete to existing concrete. Epoxies can be used also for bonding dry-pack material, fibrous concrete, polymer concretes, and some latex-modified concretes to hardened concrete. Epoxy formulations have been developed recently which will bond to damp concrete and even bond to concrete under water. There are case histories of successful uses of these materials in hydraulic structures. To help assure proper selection and use of materials, consultation with product representatives is advised before an epoxy is specified or procured. ASTM C 881 is a specification for epoxy bonding systems useful in concrete repairs, and ACI 503.2 covers epoxy bonding in repair work.

Experience has shown that the application of epoxies can create serious problems in areas of high-velocity flow. If the finished surface has a very smooth or glassy texture, flow at the boundary can be disrupted and may have the effect of a geometric irregularity which could trigger cavitation. This texture problem is easily solved by using special finishing techniques and/or improving the surface texture of the patch with sand. Sometimes the patch can be too resistant to damage, with the result that the abutting original material erodes away, leaving an abrupt change in surface geometry and developing a condition worse than the original damage.

Epoxy mortars and epoxy concretes use epoxy resins for binder material instead of portland cement. These materials are ideal for repair of normally submerged concrete, where ambient temperatures are relatively constant. They are very expensive and can cause problems as a result of their internal heat generation. Mixed results have been observed in the epoxy-mortar repair of erosion of outlet surfaces, dentates, and baffle blocks (McDonald 1980). Depending on the epoxy formulation, the presence of moisture, either on the surface or absorbed in the concrete, can be an important factor and affect the success of the repair. ACI 503.4 is a specification for epoxy mortar in repair work.

The concept of improving concrete by incorporating the epoxy directly into the mix was encouraged by the successful latex modification of concrete (Murray and Schrader 1979). Several commercial products have been developed and research is continuing. The epoxies generally enhance the concrete's resistance to freeze-thaw spalling, chemical attack, and mechanical wear. Epoxy-modified concrete (Christie, McClain, and Melloan 1981) has a curing agent which is retarded by the water in the mixture. As the water is used up by cement hydration and drying, the epoxy resin begins to gel. Accordingly, the mixture will not become sticky until the portland cement begins to set, and this greatly extends the "pot life" of the wet concrete. To date, these materials have limited use in hydraulic structures.

9.2.5 *Acrylics and other polymer systems* — There are three main ways in which polymers have been incorporated into concrete to produce a material with improved properties as compared to conventional portland cement concrete. These are polymer-impregnated concrete (PIC), polymer-portland cement concrete (PPCC), and polymer concrete (PC).

Polymer-impregnated concrete (PIC) is a hydrated portland-cement concrete that has been impregnated with a monomer which is subsequently polymerized in situ. By effectively case hardening the concrete surface, impregnation protects structures against the forces of cavitation (Schrader 1978) and abrasion erosion (Liu 1980). The depth of monomer penetration depends on the porosity of the concrete and the process and pressure under which the monomer is applied. In addition to noting that these materials are quite costly, the engineer is cautioned that some monomer systems can be hazardous and that monomer systems require care in handling and should be applied only by skilled workmen experienced in their use (DePuy 1975). Surface impregnation was used at Dworshak Dam in the repair of cavitation and abrasion erosion damage to the regulating outlet tunnels (Schrader and Kaden (1976a) and stilling basin (McDonald 1980 and Schrader and Kaden

207.3R-79	Practices for Evaluation of Concrete in Existing Massive Structures for Service Conditions
223-83	Standard Practice for the Use of Shrinkage-Compensating Concrete
302.1R-80	Guide for Concrete Floor and Slab Construction
304-73 (Reaffirmed 1983)	Recommended Practice for Measuring, Mixing, Transporting, and Placing Concrete
308-81	Standard Practice for Curing Concrete
309-72 (Revised 1982)	Standard Practice for Consolidation of Concrete
347-78 (Reaffirmed 1984)	Recommended Practice for Concrete Formwork
503R-80 (Revised 1984)	Use of Epoxy Compounds with Concrete
503.2-79	Standard Specification for Bonding Plastic Concrete to Hardened Concrete with a Multi-Component Epoxy Adhesive
503.4-79	Standard Specification for Repairing Concrete with Epoxy Mortars
506R-85	Guide to Shotcrete
506.2-77 (Revised 1983)	Specification for Materials, Proportioning, and Application of Shotcrete
544.1R-82	State-of-the-Art Report on Fiber Reinforced Concrete
544.2R-78 (Revised 1983)	Measurement of Properties of Fiber Reinforced Concrete
548R-77 (Reaffirmed 1981)	Polymers in Concrete

ASTM

A 167-84	Standard Specification for Stainless and Heat-Resisting Chromium-Nickel Steel Plate, Sheet, and Strip
C 131-81	Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
C 418-81	Standard Test Method for Abrasion Resistance of Concrete by Sandblasting
C 535-81	Standard Test Method for Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
C 779-82	Standard Test Method for Abrasion Resistance of Horizontal Concrete Surfaces
C 881-78 (Reapproved 1983)	Standard Specification for Epoxy-Resin-Base Bonding Systems for Concrete
C 884-78	Standard Test Method for Thermal Compatibility Between Concrete and an Epoxy-Resin Overlay

U.S. Army Corps of Engineers

CRD-C 63-80	Test Method for Abrasion-Erosion Resistance of Concrete (Underwater Method)

These publications may be obtained from the following organizations:

American Concrete Institute
P.O. Box 19150
Detroit, MI 48219

ASTM
1916 Race St.
Philadelphia, PA 19103

U.S. Army Corps of Engineers
U.S. Army Engineer Waterways Experiment Station
Vicksburg, MS 39180

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1976b). High-head erosion testing of PIC at Detroit Dam test facility has shown excellent performance (U.S. Army Corps of Engineers 1977).

Polymer portland cement concrete (PPCC) is made by the addition of water-dispersible polymers directly into the wet concrete mix. PPCC, compared to conventional concrete, has higher strength, increased flexibility, improved adhesion, superior abrasion and impact resistance, and usually better freeze-thaw performance and improved durability. These properties can vary considerably depending on the type of polymer being used. The most commonly used PPCC is latex-modified concrete (LMC). Latex is a dispersion of organic polymer particles in water. Typically, the fine aggregate and cement factors are higher for PPCC than for normal concrete.

A recent example of PPCC repairs was the spray application of a latex-modified fibrous concrete to a severely deteriorated navigation lock wall surface at Lower Monumental Dam on the Snake River in Washington (Schrader 1983a).

Polymer concrete (PC) is a mixture of fine and coarse aggregate with a polymer used as the binder. This results in rapid-setting material with good chemical resistance and exceptional bonding characteristics. So far, polymer concrete has had limited use in large-scale repair of hydraulic structures because of the expense of large volumes of polymer for binder. Thermal compatibility with the parent concrete should be considered before using these materials.

Polymer concretes are finding application as concrete repair materials for patches and overlays, and as precast elements for repair of damaged surfaces (Fontana and Bartholomew 1981; Scanlon 1981; Kuhlmann 1981; Bhargava 1981). Field test installations with precast PC have been made on parapet walls at Deadwood Dam, Idaho, and as a repair of cavitation and abrasion damage in the stilling basin of American Falls Dam.

ACI 548R and ACI SP-58, "Polymers in Concrete (1978)," provide an overview of the properties and use of polymers in concrete. Smoak (1985) has described polymer impregnation and polymer concrete repairs at Grand Coulee Dam.

9.2.6 *Shotcretes* — Shotcrete has been used extensively in the repair of hydraulic structures. This method permits replacing concrete without the use of formwork, and the repair can be made in very restricted areas. Shotcrete, also known as pneumatically applied mortar, can be an economical alternative to other more conventional systems of repair. ACI 506R provides guidance in the manufacture and application of shotcrete. In addition to conventional shotcrete, modified concretes such as fibrous shotcrete and polymer shotcrete have been applied by the air-blown or shotcrete method.

9.2.7 *Coatings* — High-head erosion tests have been conducted using both polyurethane and neoprene coatings (Houghton, Borge, and Paxton 1978). Both coatings exhibited good resistance to abrasion and cavitation. The problem with flexible coatings like these is their bond to the concrete surfaces. Once an edge portion of the coating is torn from the surface, th tire coating can be peeled off rather quickly by draulic force.

9.2.8 *Preplaced-aggregate concrete* — Preplacec gregate concrete, also referred to as "prepacked crete," is used in the repair of large cavities and i cessible areas (Concrete Construction Publicat 1982). Clean, well-graded coarse aggregate, general 0.5 to 1.5 in. (12 to 38 mm) maximum size, is place the form. Neat cement grout or a sanded grout, wit without admixtures, is then pumped into the aggre matrix through openings in the bottom of the forr through grout pipes embedded in the aggregate. grout is placed under pressure, and pressure is m tained until initial set. Concrete placed by this met has a low volume change because of the point-to-p contact of the aggregate; there is high bond strengt top bars for the same reason. The use of pozzol water-reducing admixtures, and low water conten recommended to further reduce shrinkage and ther volume changes, while maintaining the fluidity quired for the grout to completely fill the voids in aggregate. ACI 304-73 provides details and guid for the use of preplaced-aggregate concrete.

9.2.9 *Pipe inserts* — For repair of small-diam pipes, many of the methods discussed in the prev sections of this report are not applicable. A comr construction practice today is to obtain a jointl structurally sound pipe-inside-a-pipe without exca ing the existing unsound pipe. One such method has been used successfully is to insert a plastic pipe side the deteriorated concrete pipe and then fill the nular space between the concrete and plastic liner grout. With the proper selection of material for plastic liner pipe insert, this repair method can pro a sound, chemically resistant lining (U.S. Dept. Housing and Urban Development 1985, and U.S. vironmental Protection Agency 1983).

CHAPTER 10 — REFERENCES

10.1 — Reference standards, specifications, testing methods, and reports

The documents of the various standards-produc organizations referred to in this document are lis below with their serial designation, including year adoption or revision. The documents listed were latest effort at the time this document was writt Since some of these documents are revised frequen generally in minor detail only, the user of this do ment should check directly with the sponsoring grc if it is desired to refer to the latest revision.

American Concrete Institute

117-81	Standard Tolerances for Conc Construction and Materials
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