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College of Engineering and Technology

Department of Civil and Architectural Engineering

Graduation project:

Structural Design of Training Center Units

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التصميم الإنشائي لوحدات مركز تدريب

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جامعة بوليتكنك فلسطين

٢٠١٣- ٢٠١٤ م

إشراف :

د. محمد طه السيد احمد

ملخص المشروع

يهدف المشروع الى اكتساب القدرة والمعرفة في تصميم وحدات في مركز تدريب تتضمن مظلة خرسانية لوقوف السيارات، مطعم وكافتيريا، مبنى اداري، وبركة سباحة. سوف يتم التصميم يدويا وايضا باستخدام البرامج وذلك حسب الكود الامريكي في التصميم

(ACI 318-08)

فيما سيتم استخدام كود الأحمال الأردني لحساب الأحمال.

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List of Abbreviations

As = area of tension reinforcement.

Ag = gross area of section.

b = width of compression face of member.

bw = web width, or diameter of circular section.

dl = dead loads.

d = distance from extreme compression fiber to centroid of tension reinforcement.

Ec = modulus of elasticity of concrete.

fc' = compression strength of concrete.

fy = specified yield strength of non-prestressed reinforcement.

h = overall thickness of member.

LL = live loads.

M = bending moment.

Mu = factored moment at section.

Mn = nominal moment.

Pn = nominal axial load.

Pu = factored axial load

Vc = nominal shear strength provided by concrete.

Vn = nominal shear stress.

Vs = nominal shear strength provided by shear reinforcement.

Vu = factored shear force at section.

W = width of beam or rib.

Wu = factored load per unit area.

Φ = strength reduction factor.

εc = compression strain of concrete = 0.003mm/mm.

εs = strain of tension steel.

έs = strain of compression steel.

ρ = ratio of steel area.

١. مقدمة عامة عن المشروع

١.١ مقدمة

تعتبر الهندسة المدنية أحد فروع الهندسة الأساسية والتي يتسع مجالها ليشمل العديد من مناحي الحياة في العصر الحالي، فهي تعالج مشاكل تصميم وتحليل المنشآت المدنية مثل المساكن والمدارس والمستشفيات، وتعالج أيضاً مشاكل البنية التحتية للمدينة المعاصرة والتي تشمل شق الطرق وبناء الجسور والمطارات والسكك الحديدية وإدارة المرور وشبكات المياه والصرف الصحي، ومن المجالات التي تعالجها الهندسة المدنية أيضاً مشاكل البيئة والحفاظ عليها مثل طرق معالجة مياه الصرف الصحي ودراسة التلوث البيئي وطرق تخفيف حدته، هذا بالإضافة إلى بناء السدود وحفر الآبار والقنوات وإدارة المشاريع الهندسية وإدارة النفايات الصلبة، والمهندس المدني هو ذلك المتخصص الذي يؤهل للتصدي لتلك المشاكل محاولاً إيجاد الحلول المناسبة والاقتصادية لها.

وتعتبر هندسة الإنشاءات مجال هندسي يتعامل مع تصميم المنشآت التي تدعم أو تقاوم الأحمال .و تعنى الهندسة الإنشائية بدراسة التحليلات النظرية والتصاميم لكافة أنواع المنشئات وتطبيقاتها آخذين بنظر الاعتبار كافة التأثيرات الاستاتيكية والديناميكية وعلاقتها بكافة تأثيرات البيئة من رياح وزلازل وظروف الطقس المختلفة.

١.٢ تعريف بالمشروع

تعد مراكز التدريب مهمة للمتابعة وتطوير الموظفين وتلبية حاجاتهم الضرورية (تنمية الموارد البشرية) التي تسهل، وتسرع في تحقيق الإنجاز الجيد في العمل، ويكون ذلك من خلال البرامج التدريبية التي يحتاجها كل موظف حسب مجال عمله.

والمشروع عبارة عن تصميم وحدات معينة في مركز تدريب يقع في مدينة بيت لحم والذي يتضمن تصميم مظلة خرسانية لمواقف السيارات وتصميم مبنى ادارية يتكون من اربعة طوابق وكافتيريا بالإضافة الى تصميم بركة سباحة تتكون من ثلاث مستويات وسيأتي على ذكرها بالتفصيل.

١.٣ أهداف المشروع

التركيز على فكرة القدرة على اختيار النظام الانشائي الملائم للطبيعة الموجودة في الوحدات المرد تصميمها.
العمل على ايجاد العلاقة بين المسقات النظرية المطروحة والواقع العملي الذي يحاك بتلك المساقات النظرية.
اكتساب خبرات ومهارات جديدة تنتج من مشاكل تواجه فريق العمل لم يتطرق لها من قبل من خلال الدراسة النظرية.
يعد المشروع مرجع متكامل في التحليل والتصميم للعناصر الانشائية المختلفة.

١.٤ مشكلة المشروع

تتمثل مشكلة المشروع في التحليل والتصميم الانشائي للعناصر الانشائية المكونة للوحدات المراد تصميمها وعليه سيتم تحليل كل عنصر من العناصر الانشائية المعروفة من جسور وأعمدة وغيرها ، من خلال عملية تحديد الاحمال الواقعة على كل عنصر انشائي من اجل تحديد الابعاد والتسليح الذي يضمن الامان والعمل بكفاءة لذلك العنصر المصمم ،ثم عملية ايجاد المخططات التنفيذية لتلك العناصر التي تم تصميمها بالاعتماد على الاسس الصحيحة بصورة تضمن اخراج هذا البحث بصورة متكاملة تخرج من حيز الاقتراحات الى حيز التطبيق.

١.٥ المخطط الزمني لمراحل العمل بالمشروع

المخطط الزمني لمراحل العمل بالمشروع وفق الخطوات المقترحة خلال 32 أسبوع :

جدول (١-١): المخطط الزمني لمراحل العمل بالمشروع

مرحلة الزمنية المقترحة (بالاسبوع)	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
اختيار المشروع																																
دراسة الموقع																																
جمع المعلومات حول المشروع																																
دراسة المشروع معماريا																																
دراسة المشروع انشائيا																																
تصميم المظلة الخرسانية (يدويا)																																
تصميم عصب للمبنى الاداري																																
اعداد مقدمة المشروع																																
عرض مقدمة المشروع																																
التحليل الانشائي لكافة المباني المتبقية																																
التصميم الانشائي لكافة المباني المتبقية																																
اعداد المخططات																																
كتابة المشروع																																
عرض المشروع																																

٢. الوصف المعماري

٢.١ لمحة عامة عن المشروع

يتكون مشروع المركز التدريبي اساسا من العديد من الوحدات ولكننا في هذا المشروع اخترنا تصميم اربع وحدات فقط وهي مظلة خرسانية لمواقف السيارات و مبنى إداري يتكون من اربعة طوابق وكافتيريا بالإضافة الى بركة سباحة واسعة تتكون من ثلاث مستويات وهذه العناصر سيتم توضيحها في هذا الفصل.

٢.٢ موقع المشروع

تقع قطعة الأرض المقترحة لإقامة المشروع عليها على شارع القدس – الخليل في وسط مدينة بيت لحم في منطقة باب الزقاق.

الشكل (٢-١): صورة جوية لقطعة الارض

٢.٣ مساحة قطعة الارض

تبلغ مساحة قطعة الأرض المقترحة لإقامة المشروع حوالي ٩٠٠٠ متر مربع.

٢.٤ اسباب اختيار الموقع

مساحة الأرض المتناسبة مع مساحة وحجم المشروع.

طبيعة الارض السهلة وهذا يساعد في عمليات الحفر وعمل الاساسات.

طبيعة المنطقة متناسبة مع طبيعة المبنى وأغراضه.

منطقة رئيسية تجارية تقع على شارع رئيسي وهذا يسهل الوصول اليها.

عدم وجود طابع معماري موحد في المنطقة يجعل من المبنى عنصراً متميّزاً في المنطقة ككل.

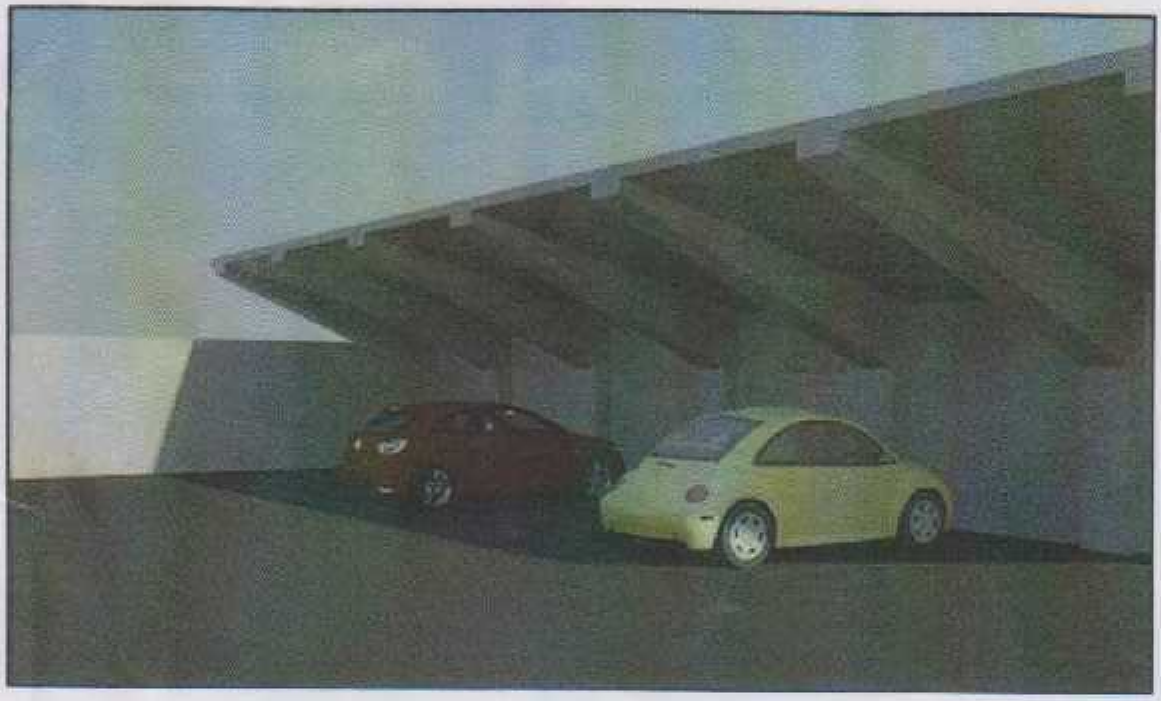
٢.٥ حركة الرياح في الموقع

تؤثر الرياح بشكل كبير على المبنى سواء على الجدران او الهيكل الانشائي بالإضافة الى عمليات الحت والتعرية لذا فان مراعاة اتجاه الرياح عند توجيه المبنى امر ضروري في عملية التصميم وعادة ما يختلف هبوب الرياح وسرعتها من منطقة لأخرى ومن فصل لأخر، فالعوامل المحلية من ضغوط وتضاريس لها اثر كبير في اختلاف الريال التي تنتظم في خطوط عامة واهم الرياح التي تهب على مدينة بيت لحم وتؤثر على الموقع المقترح هي الرياح الجنوبية الشرقية التي تهب شتاء والرياح الشمالية الغربية التي تهب صيفاً وشتاءً لذا يجب الانتباه جيدا عند توجيه المبنى.

٢.٦ وصف عناصر المشروع

٢.٦.١ مظلة السيارات

عبارة عن مظلة خرسانية بامتداد عرضي معلق مقداره خمسة امتار وارتفاعها مترين ونصف وتضم العديد من مواقف السيارات بين كل موقف والاخر ثلاثة امتار وتقع على السور الخارجي المحيط بمركز التدريب كما هو موضح في الصورة.



الشكل (٢-٢): مظلة السيارات

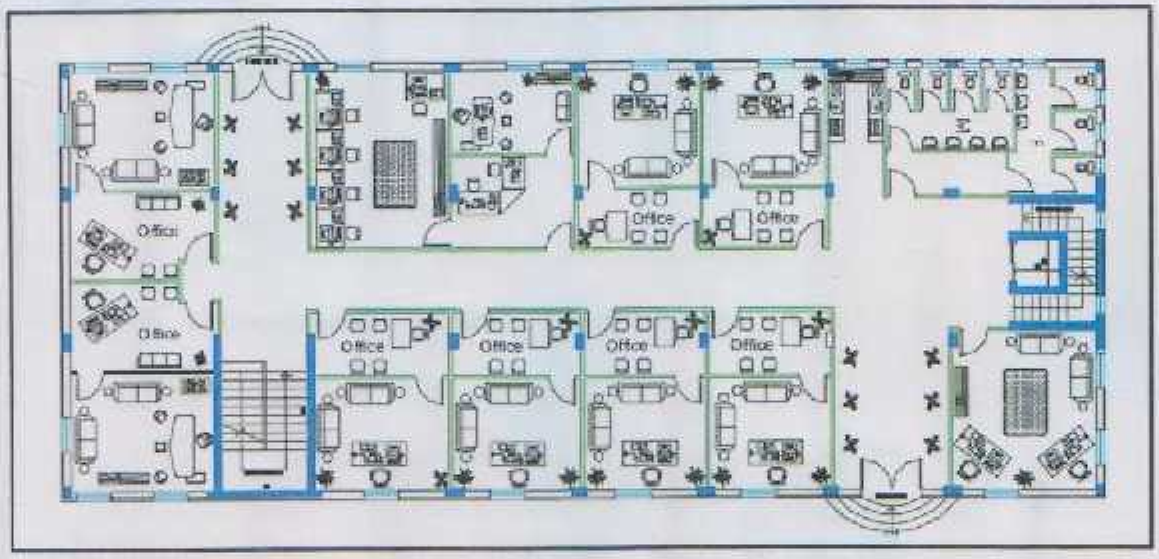
٢.٦.٢ المبنى الاداري

الشكل (٢-٣): المبنى الإداري

يتكون المبنى الاداري من اربعة طوابق مفصلة كما يلي :

٢.٦.٢.١ الطابق الأرضي

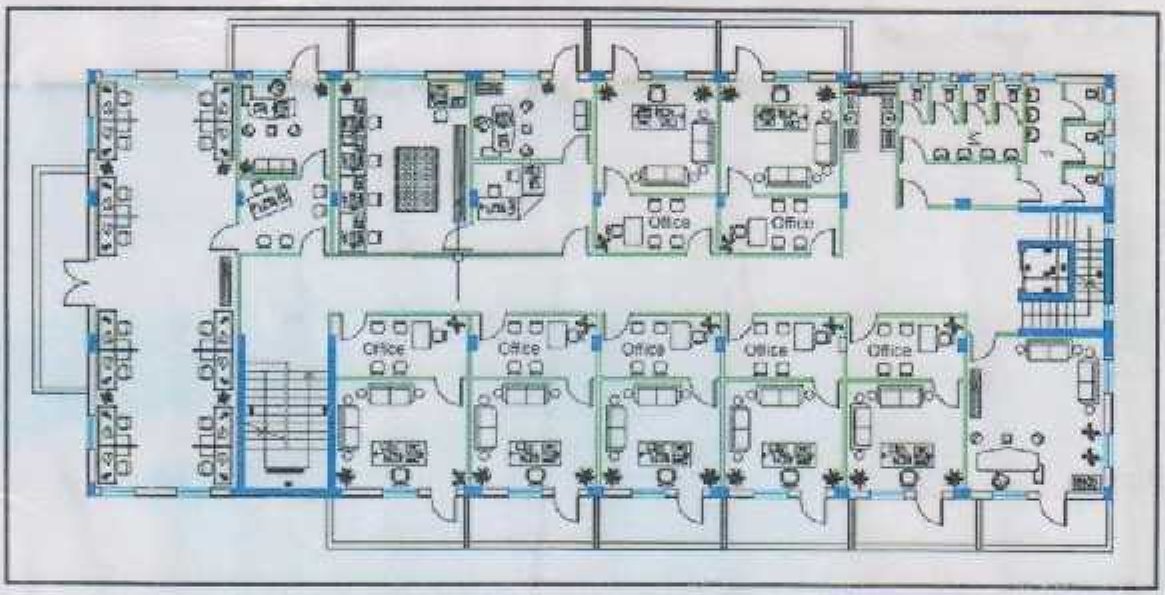
تبلغ مساحة هذا الطابق حوالي ٥٣٠ م٢ ،ويتكون من أقسام وأجزاء خدماتية متنوعة ،حيث أنه يحتوي العديد من المكاتب ودرجين رئيسين ومصعد واحد ، ودورات مياه خاصة بالرجال والنساء.



الشكل (٢-٤): الطابق الأرضي

٢.٦.٢.٢ الطابق الأول حتى الرابع

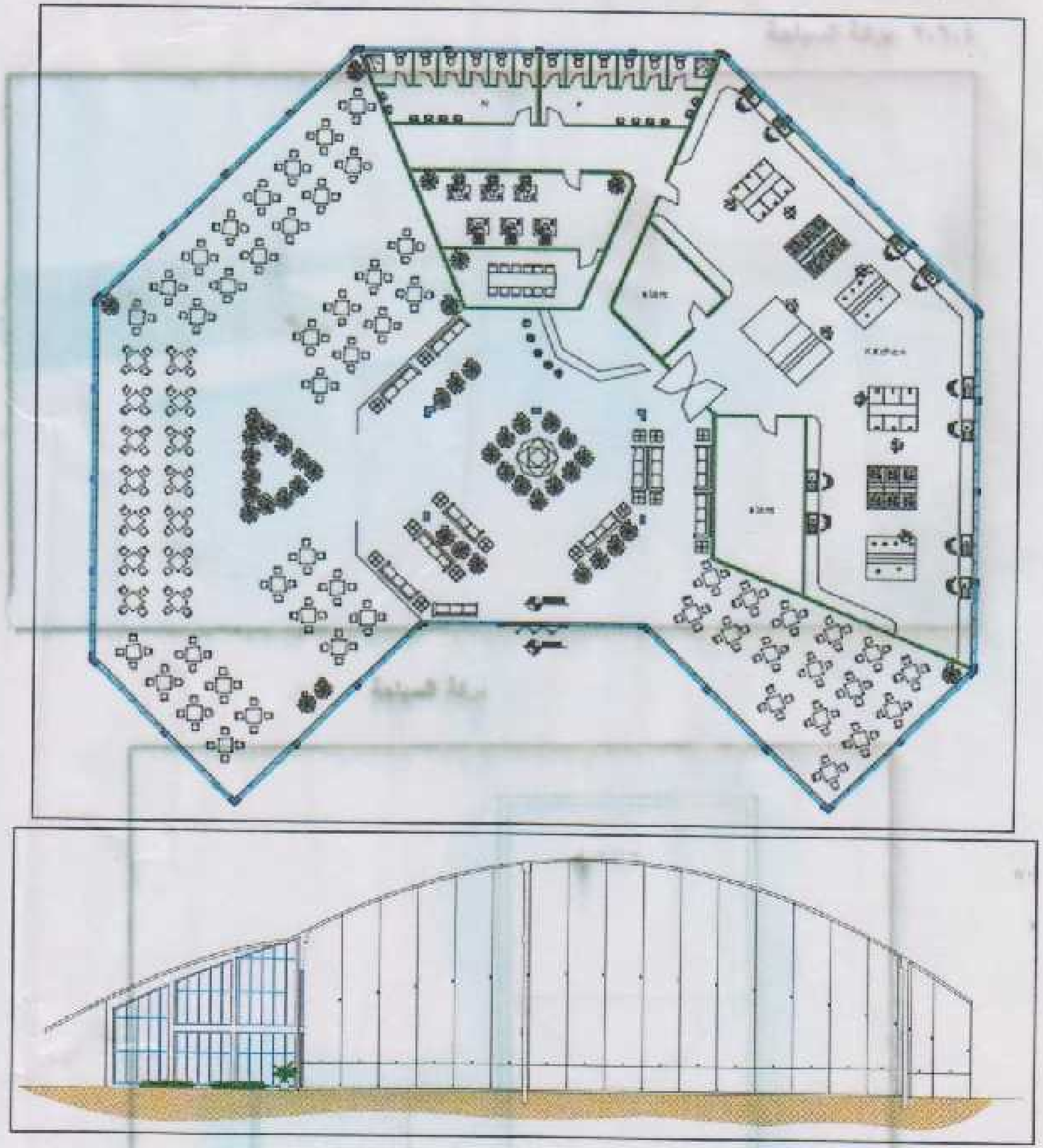
تبلغ مساحة هذا الطابق المكرر حوالي ٦٠٧ م٢ ،ويتكون الطابق من مجموعة من المكاتب الخدماتية ومكاتب الموظفين والإداريين و رؤساء الأقسام ومكتب صحافة ،ويتكون أيضا من دورة مياه ومطبخ .



الشكل (٢-٥): الطابق الأول حتى الرابع

٢.٦.٣ الكافتيريا

الشكل (٢-٦): مبنى الكافتيريا

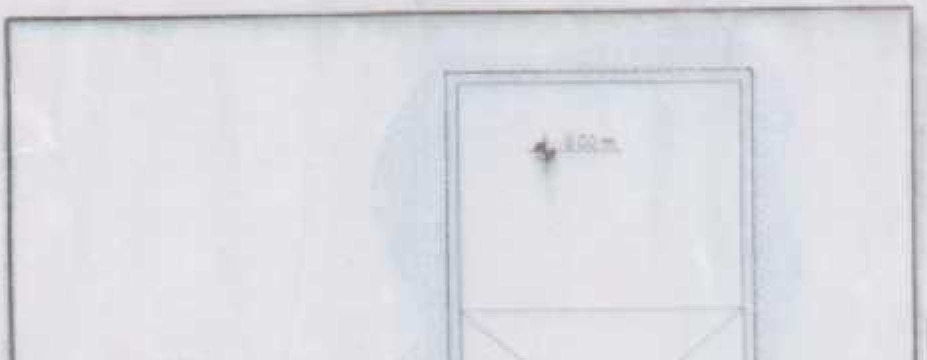


الشكل (٢-٧): مخطط افقي ومقطع عرضي للكافتيريا

تبلغ مساحة الكافتيريا حوالي ٢٤٠٠ م^2 ، وهي عبارة عن مطعم كبير بشكل ثماني الاضلاع غير منتظم تتراوح حقوله حوالي ٢٥ متر والتي سيتم تحليلها وتصميمها خلال المشروع في الفصل القادم ان شاء الله .

٢.٦.٤ بركة السباحة

بركة السباحة



مخطط افقي لبركة السباحة

تبلغ مساحة بركة السباحة حوالي ٤٤٠ م٢ وهي تتكون من ثلاث مستويات المستوى الاول ، المستوى الاول بعمق ١.٥ م والمستوى الثاني بعمق ٣ والمستوى الثالث بعمق ٦ امتار .

٣. الوصف الانشائي

٣.١ الهدف من التصميم الإنشائي

إن الهدف من التصميم الإنشائي هو إنتاج مبنى آمن ، مترابط يلبي الهدف من إنشائه ، و يحقق جميع المتطلبات الهندسية و الإنشائية و مقاوم للمؤثرات الخارجية البيئية من زلازل و رياح و هبوط للتربة . فعند تصميم أي عنصر إنشائي لا بد من مراعاة المعايير التالية :

الأمان (Safety) : و يتم تحقيقه من خلال اختيار العنصر الإنشائي المناسب في المكان المناسب ، القادر على مقاومة الأحمال التي يتعرض لها بأمان .

التكلفة (Cost) : و يتم تحقيقها من خلال مواد البناء المستخدمة ، و مقاطع مناسبة التكلفة و كافية للغرض الذي ستستخدم لأجله ، من دون المبالغة فيها .

حدود صلاحية المبنى للتشغيل (Serviceability) : من حيث تجنب حدوث أي هبوط زائد و تجنب التشققات التي تشوه المبنى معمارياً ، و تضعفه إنشائياً .

الرونق الجمالي للمبنى .

٣.٢ الأحمال و أنواعها

الأحمال هي قوى يتم تطبيقها على عنصر من عناصر المنشأ أو على المنشأ ككل باعتباره وحدة واحدة ، و عملية تحديد هذه الأحمال عملية معقدة ، حيث أن طبيعة هذه الأحمال تختلف باختلاف التصميم المعماري ، المواد الإنشائية المستخدمة و موقع المنشأ .

و تقسم الأحمال حسب طريقة تأثيرها بالمنشأ إلى :

- الأحمال الرئيسية : تتضمن الأحمال الميتة ، الحية و البيئية .

الأحمال الثانوية : تتضمن انكماش الخرسانة ، التأثير الحراري ، الزحف و هبوط الأساسات .

إن تقدير الأحمال الواقعة على المنشأ من واجبات المهندس الأساسية فمن الواجب مراعاة الدقة في تمثيل هذه الأحمال على العناصر الإنشائية ، حيث أن الخطأ في الحسابات الإنشائية قد يؤثر سلبا على المنشأ و يتسبب في خسائر مادية و بشرية فادحة و هذه الأحمال هي :

٣.٢.١ الأحمال الميتة

إن المنشأ و قبل كل شيء يحمل الأحمال الميتة و التي تتضمن الوزن الذاتي للعناصر الإنشائية و التجهيزات الثابتة التي يتكون منها ، وهذه أحمال ثابتة من حيث المقدار و الموقع . و يمكن حسابها من خلال تحديد أبعاد للعنصر الإنشائي و الكثافة النوعية للمواد المكونة له ، و الجدول التالي يبين الكثافة النوعية للمواد المستعملة في عملية البناء :

جدول (٣-١): الكثافة النوعية للمواد المستخدمة في البناء

الرقم المتسلسل	المادة المستخدمة	الكثافة المستخدمة (KN/m³)
١	البلاط	23
٢	المونة	22
٣	الرمل	17
٤	القصارة	22
٥	الخرسانة المسلحة	25
٦	كلكل للعزل في الجدران بسماكة 5cm	0.2
٧	طوب للجدران بسماكة 10 cm	10
٨	طوب للعقدات بسماكة 24 cm	15

٣.٢.٢ الأحمال الحية

وتتضمن أوزان الأشخاص ، الأثاث ، الأجهزة و المعدات ، المواد المخزنة و السيارات ، و هذه أحمال متغيرة من حيث المقدار والموقع بصورة مستمرة ، و تعتمد قيمتها على طبيعة استخدام المنشأ او جزء منه.

٣.٢.٣ الأحمال البيئية

و هي الأحمال الناتجة عن التغيرات البيئية التي يتعرض لها المنشأ كأحمال الزلازل ، الرياح و الثلوج.

٣.٢.٣.١ أحمال الزلازل

تحدث الزلازل بسبب الحركة المفاجئة لصفائح التكتونية المكونة للقشرة الأرضية على طول الصدوع الموجودة مسبقاً ، حيث أنها تسبب اهتزازات أفقية و عمودية ينتج عنها عزوم منها عزم الانقلاب و عزم الدوران و يتم مقاومتها بجدران القص المصممة بسماكات و تسليح كاف يضمن سلامة المنشأ عند تعرضه لمثل هذه الأحمال التي يجب مراعاتها في عملية التصميم لتقليل الخطورة و المحافظة على أداء المنشأ لوظيفته أثناء الزلزال . و نظراً لصعوبة تحليلها فإنه يتم تحويل أحمال الزلازل إلى قوى ثابتة من خلال أخذ قيم مكافئة لها.

٣.٢.٣.٢ أحمال الرياح

و هي قوى أفقية تؤثر في المبنى و يظهر تأثيرها في المباني المرتفعة ، و تكون موجبة إذا كانت ناتجة عن ضغط و سالبة إذا كانت ناتجة عن شد و تقاس ب KN/m^2 . و يتم تحديدها اعتماداً على سرعة الرياح القصوى ، ارتفاع المبنى و موقعه من حيث الإحاطة بمبانٍ أخرى سواء كانت مرتفعة أو منخفضة.

و قد أصبحت هذه الأحمال مهمة جداً في السنوات الأخيرة بسبب الاستخدام الواسع للمواد الأخف وزناً و تقنيات البناء الجديدة التي مكنت من بناء مبانٍ أكثر ارتفاعاً من ذي قبل .

٣.٢.٣.٣ أحمال الثلوج

و هي الأحمال الناتجة بفعل تراكم الثلوج و يمكن تقييمها بناءً على الأسس التالية :

ارتفاع المنشأ عن سطح البحر .

ميلان السطح المعرض لتساقط الثلوج .

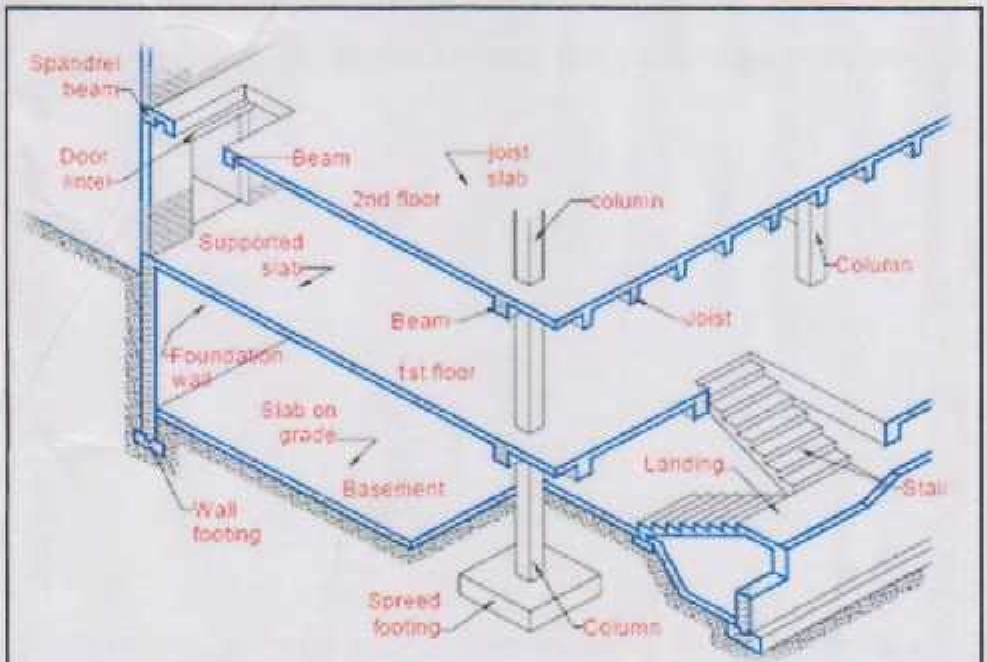
و الجدول التالي يبين قيمة أحمال الثلوج بناءً على ارتفاع المنشأ عن سطح البحر حسب الكود الأردني :

جدول (٣-٢): أحمال الثلوج بناءً على ارتفاع المنشأة عن سطح البحر حسب الكود الأردني

الارتفاع عن سطح البحر "h" (المتر)	احمال الثلوج (KN/m^2)
h < 250	0
500 > h > 250	(h-250)/1000
1500 > h > 500	(h-400) / 400
2500 > h > 1500	(h – 812.5)/ 250

٣.٣ العناصر الإنشائية المستخدمة

هناك مجموعة من العناصر الإنشائية التي تعمل معاً كوحدة واحدة لمقاومة الأحمال الواقعة عليها و من أهم هـذه العناصر : البلاطات الخرسانية ، الجسور ، الأعمدة ، الأدراج ، الجدران الحاملة و الأساسات .



الشكل (3–1): بعض العناصر الإنشائية المكونة للمنشأة

٣.٣.١ البلاطات الخرسانية (Slabs)

و هي العناصر الإنشائية التي تقوم بنقل الأحمال الواقعة عليها إلى العناصر الإنشائية الحاملة لها في المنشأ مثل الجسور و الأعمدة و الجدران . و يتم اختيار النوع الأمثل بالاعتماد على عدة عوامل أهمها :

الفضاءات بين الأعمدة .

وظيفة المنشأ .

التكلفة.

السهولة ، الوقت ، القوالب الشائعة منها .

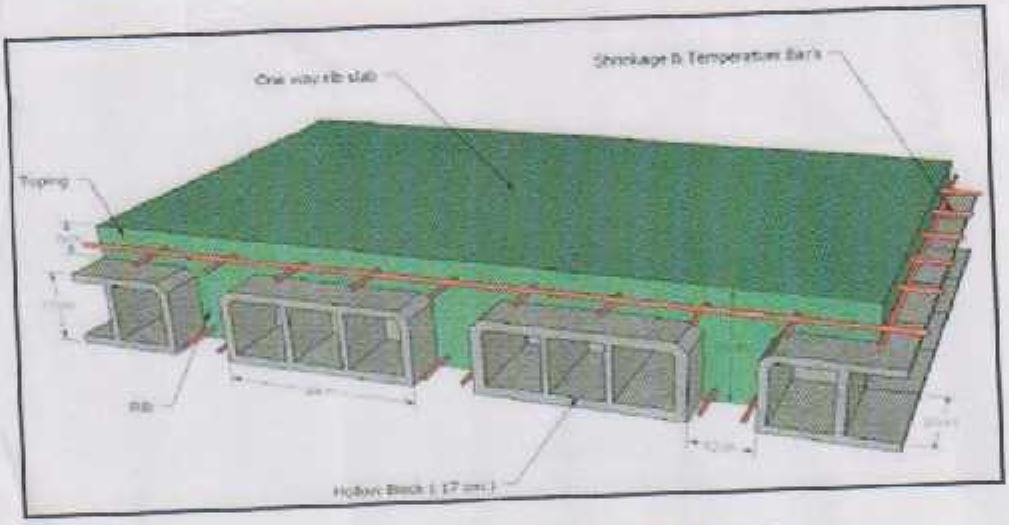
ونظراً لوجود العديد من الفعاليات في المشروع ،و تنوع المتطلبات المعمارية فإنه تم استخدام الأنواع التالية حسبما هو ملائم لطبيعة الاستخدام :

Ribbed slabs
Solis slabs

٣.٣.١.١ بلاطات العصب (Ribbed Slab)

وقد تم استخدام نوع بلاطة العصب ذات الاتجاه الواحد . حيث تتكون هذه البلاطة من أعصاب فوقها بلاطة تغطية سماكتها ما بين (5-10cm) تصب فوق قوالب مؤقتة أو دائمة ، و قد تكون الجسور فيها مسحورة أو ساقطة .

و تسمى بلاطة عصب ذات الاتجاه الواحد (One Way Ribbed Slabs) إذا كانت الأعصاب باتجاه واحد فقط و يتم استخدامها إذا كانت النسبة بين طول البلاطة و عرضها أكبر من 2 .

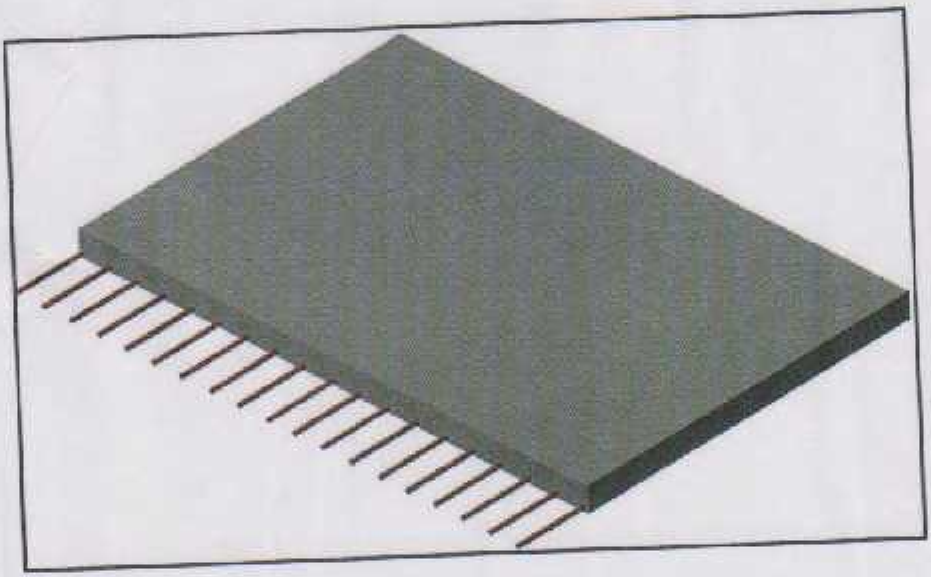


الشكل (3−٢): بلاطة العصب ذات الاتجاه الواحد

٣.٣.١.٢ البلاطات المصمتة (Solid Slabs)

و قد تم استخدام البلاطات المصمتة ذات الاتجاه الواحد (One Way Solid Slabs) في المشروع .

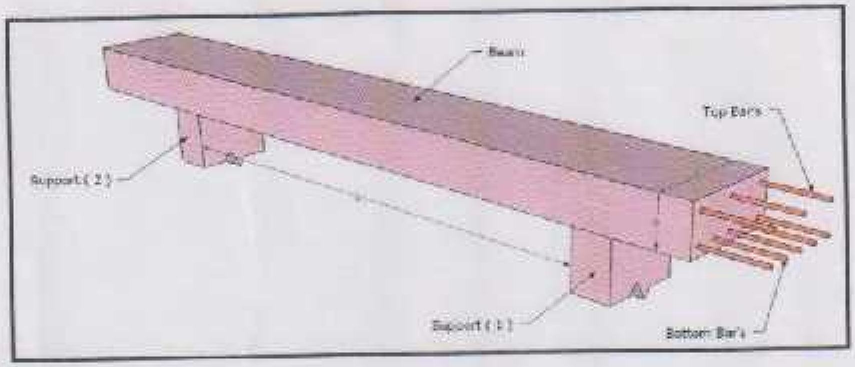
و تسمى بذلك إذا كان الإسناد من جهتين فقط حيث تنتقل الأحمال باتجاه الإسناد و منها للأعمدة أو إذا كان الإسناد من الجهات الأربعة لكن الاتجاه الطويل أكبر من ضعف القصير ففي هذه الحالة تنتقل الأحمال بالاتجاه القصير، و تكون الجسور فيها من نوع (Dropped Beams).



الشكل (3−٣): بلاطة المصمتة

٣.٣.٢ الجسور (Beams)

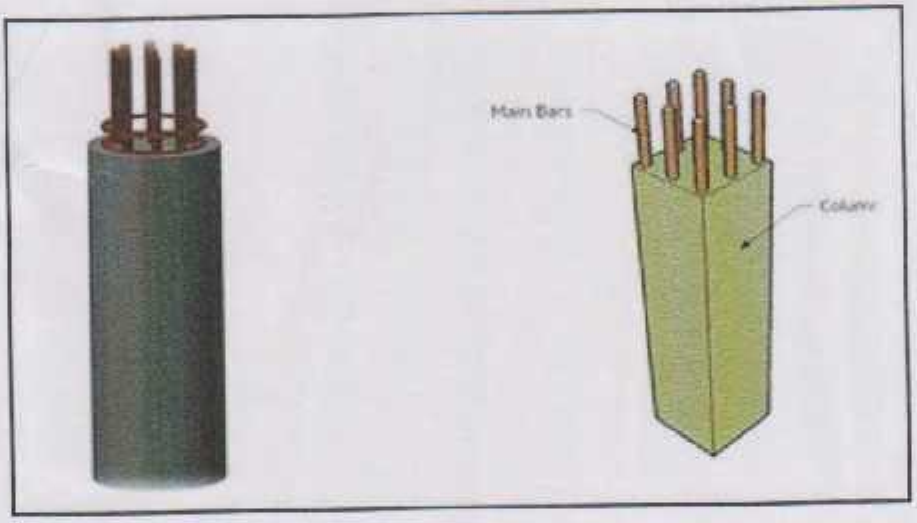
و هي عناصر إنشائية أساسية تقوم بنقل الأحمال من العقدات إلى الأعمدة ، و قد تكون مسحورة (Hidden Beam) أي مخفية داخل العقدة و لها نفس ارتفاع العقدة ، أو ساقطة (Dropped Beam) أي ان ارتفاعها أكبر من ارتفاع العقدة، و قد يتم إبراز الجزء الزائد من الجسر في أحد الاتجاهين العلوي أو السفلي .



الشكل (3-٤): جسر مسحور

٣.٣.٣ الأعمدة (Columns)

و هي العناصر الإنشائية الرئيسية التي تقوم بنقل الأحمال من العقدات و الجسور إلى الأساسات ، و بذلك فهي عنصر رئيسي في نقل الأحمال و ثبات المنشأ و يجب تصميمها بحيث تكون قادرة على نقل و توزيع الأحمال الواقعة عليها . و بالنسبة لأنواعها فمنها الطويلة و القصيرة ، و حسب أشكالها فهناك الدائرية و المستطيلة و المربعة .

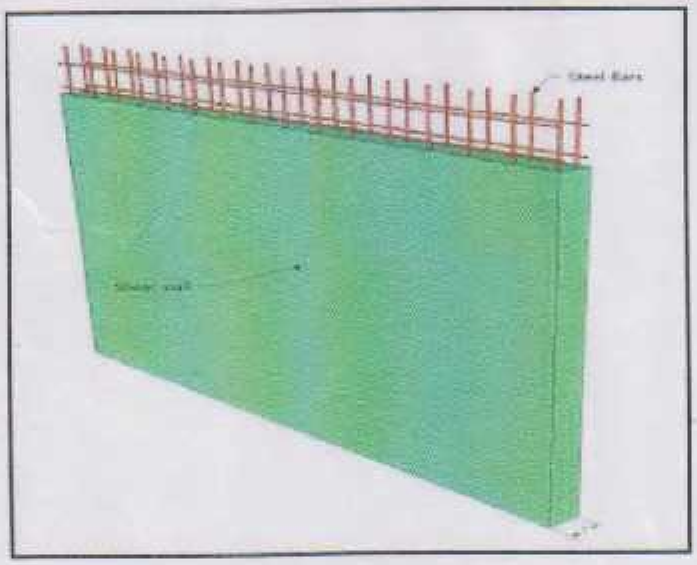


الشكل (3-٥): بعض أنواع الاعمدة المسلحة

٣.٣.٤ جدران القص (Shear Walls)

و هي عناصر إنشائية حاملة تقاوم القوى الأفقية و العمودية الواقعة عليها و تستخدم بشكل رئيسي في مقاومة الأحمال الأفقية كأحمال الرياح و الزلازل ، و تسلح بطبقتين من الحديد لزيادة كفاءتها في مقاومة الأحمال الأفقية الواقعة عليها .

تعمل هذه الجدران على تحمل الأحمال الرأسية المنقولة إليها كما تعمل على مقاومة القوى الأفقية التي تتعرض لها المنشأ ، و يجب توفرها في الاتجاهين مع مراعاة أن تكون المسافة بين مركز المقاومة الذي تشكله جدران القص في كل اتجاه و مركز ثقل المنشأ أقل ما يمكن ، و أن تكون هذه الجدران كافية لتقليل تولد العزوم و آثارها على جدران المنشأ المقاومة للقوى الأفقية . و تتمثل هذه الجدران بجدران بيت الدرج و جدران المصاعد و الجدران الأخرى التي تبدأ من الأساسات .



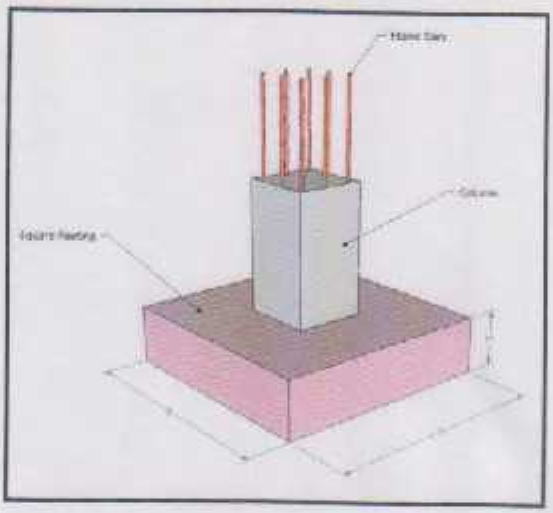
الشكل (3-٦): جدران القص

٣.٣.٥ الأساسات (Foundations)

بالرغم من أن الأساسات هي أول ما نقوم بتنفيذها إلا أنها آخر ما نقوم بتصميمه . و تعتبر الأساسات حلقة الوصل ما بين العناصر الإنشائية في المنشأ و الأرض .

و لمعرفة الأحمال و الأوزان الواقعة عليها ، فإن الأحمال الواقعة على العقدة تنتقل إلى الجسور منها إلى الأعمدة و أخيرا إلى الأساسات التي تقوم بنقلها و توزيعها في التربة . بالتالي يكون الأساس مسؤول عن تحمل الأحمال

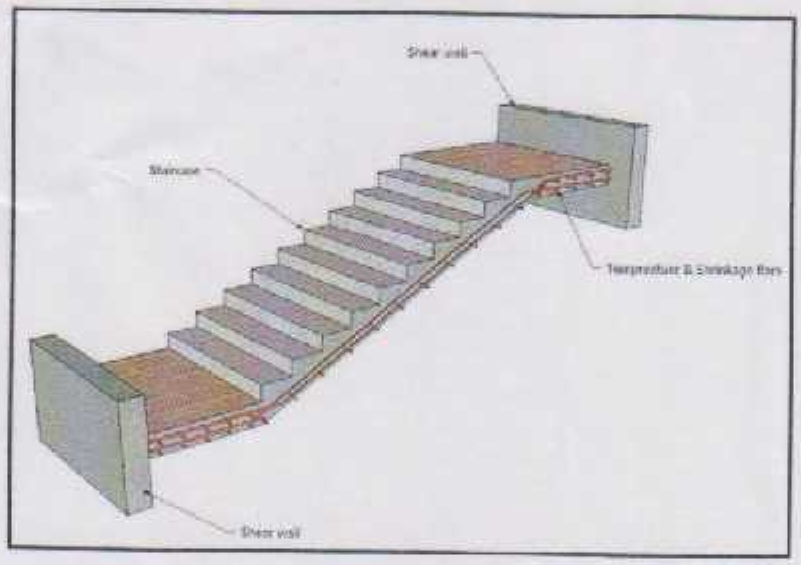
الميتة و الحية للمنشأ بالإضافة لأحمال الزلازل و الرياح و الثلوج . و تكون هذه الأحمال هي الأحمال التصميمية للأساسات و بناءً عليها و على طبيعة التربة يتم تحديد أبعاد و نوع الأساسات المستخدمة ، وقد تم استخدام أساسات من أنواع مختلفة و ذلك تبعاً للأحمال الواقعة على كل أساس و قوة تحمل التربة تحته .



الشكل (3-٧): الأساسات المنفردة

٣.٣.٦ الأدراج (Stairs)

و هي العناصر المسؤولة عن الانتقال الرأسي بين الطبقات في المنشأ ، و قد تم تصميم الدرج إنشائياً باعتباره عقدة مصمتة في اتجاه واحد .



الشكل (3-٨): مقطع في الدرج

٣.٤ برامج الحاسوب التي سوف تستخدم

1. AutoCAD Structural detailing .
2. For Text Edition Microsoft Office.
3. Atir Software for Structural Calculations.
4. Safe.
5. Etabs.
6. Autodesk Robot structural analysis program.
7. Autodesk Rivet Architecture.
8. Column Expert

٣.٥ الوصف الانشائي لوحدات المشروع

٣.٥.١ مظلة السيارات

النظام الانشائي المستخدم للمظلة هو الاطار (frame) بنظام ال cantilever بطول تعليق قدره ٥ امتار وارتفاع النظام وصولا الى القاعدة يساوي ٣.٥ امتار وبمسافة ٣ متار بين كل (frame) والذي يليه , وهناك عقدة مصمتة (Solid Slab) باتجاه تحميل واحد بين كل اطار والاخر، و القاعدة تقوم بنقل الاحمال الرأسية والعزم الى الارض لأنه نظمها ثابت (fixed) ، وتم اخذ ثلاث حقول من المظلة وتصميم كافة عناصرها لأجل التبسيط.

٣.٥.٢ مبنى الاداري

يتكون المبنى الاداري من عقدات الاعصاب بالإضافة الى البلاطات المصمتة في عقدات بيت الدرج و الاعمدة وجدران القص والاساسات ، والتي تم تصميمها باستخدام البرامج والحسابات اليدوية.

٣.٥.٣ الكافتيريا

نظرا لشكل المبنى المعقد المتماثل حول محور واحد وبسبب رغبة المالك بعدم وضع اعمدة في الفضاء الداخلي للمبنى ، فقد تم استخدام عقدة قشرية خرسانية تتناسب مع شكل المبنى بسماكة ١٥ سم ، والمحملة على جسور مقوسة تتلائم مع شكل العقدة .

٣.٥.٤ بركة سباحة

تم تصميمها بناء على القوة الداخلية الناتجة من الاحمال الخارجية المتولدة من ضغط التربة والمياه ، وتم الاخذ بعين الاعتبار بحالات التحميل التي تتعرض لها هذه البركة.

4. Structural Design

4.1 Design of Parking Sheds

As stated before the structural system of the parking shed is frame system that carried a one way solid slab 14 cm thick. The design process includes the design of intermediate frame and the exterior and theirs footings.

For all of the element of the shed the concrete compression strength (f_c') is 28 Mpa and the tensile strength of reinforcement rebar (f_y) is 414 Mpa, and the density of concrete is 25 KN/m^3.

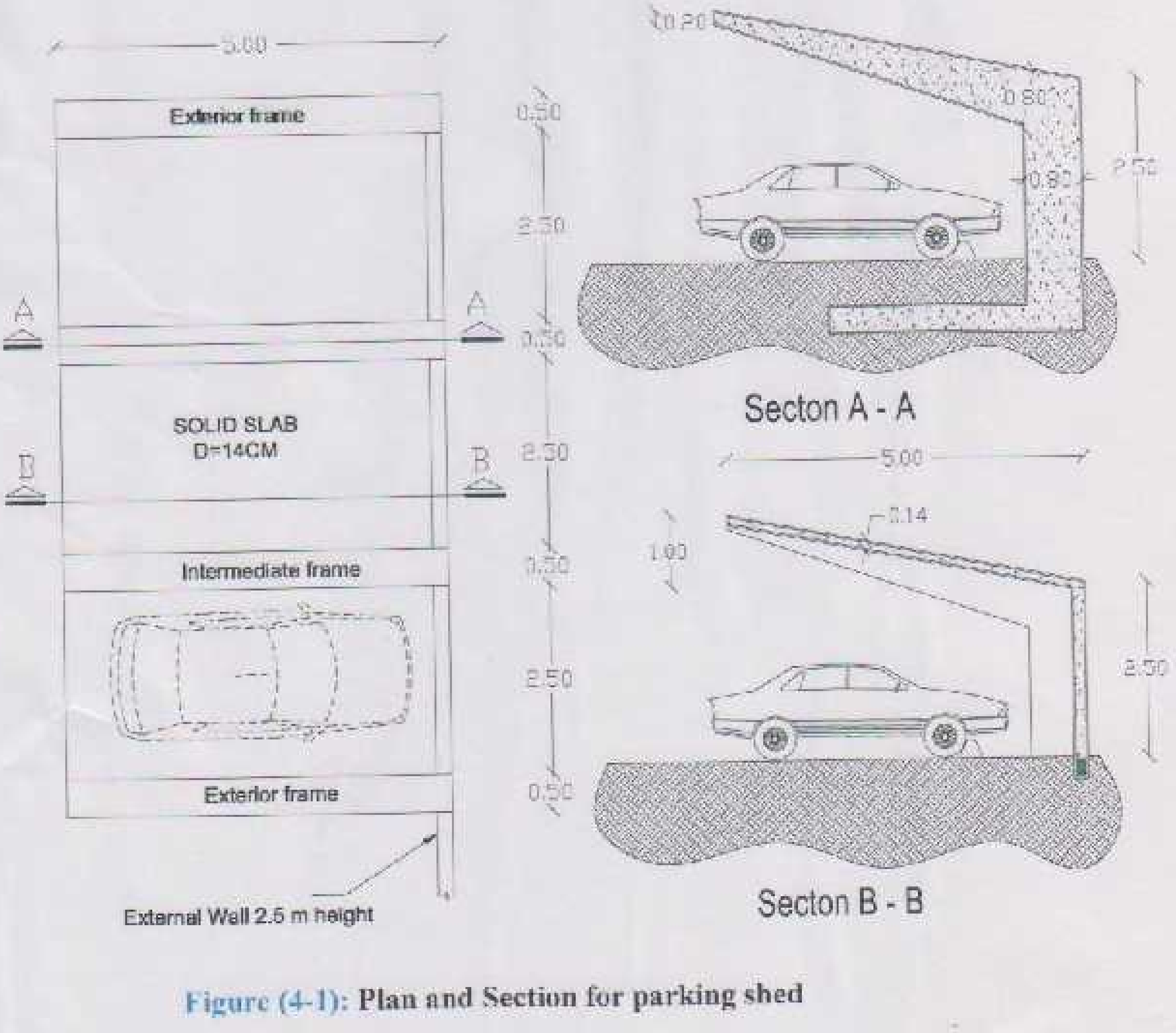


Figure (4-1): Plan and Section for parking shed

4.1.1 Design of Slab

Slab will be one-way solid slab.

Thickness of slab will be selected so that **No Shear Reinforcement** is required. and it's calculated according to provisions in **ACI-318-08** as follow:

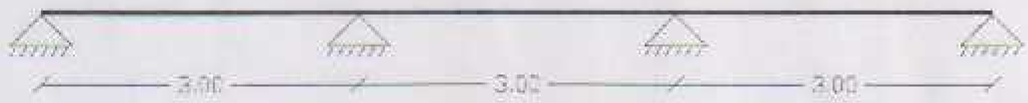


Figure (4-2): **System of slab of parking shed**

One end Continues: .(ACI-318-08 In table 9.5(a))

$h_{min} = L / 24$

$= 300 / 24 = 12.5$ cm

Both end Continues: .(ACI-318-08 In table 9.5(a))

$h_{min} = L / 28$

$= 300 / 28 = 10.7$ cm

Select h = 14 cm.

4.1.1.1 Calculation of loads

For 1 m strip

Dead load of slab (self-weight)

$0.14 \times 25 \times 1 = 3.5$ KN/m

Live Load

[1 KN/m^2] will be applied to the slab

1 KN/m$^2 \times 1 = 1$ KN/m

Snow load

Because of the risk of Accumulation of the snow on the slab of the shed a depth of 1m of ice will be considered:

[According to Jordanian code 2/8/3 page 62]

$1 \times$ [10 KN/m^3] $\times 1 = 10$ KN/m

Wind load

according to Jordanian code the wind load will be calculated as follows :

Basic wind speed V = 35 m/s [assumed]

Design speed:-

$V_Z = V \times S_1 \times S_2 \times S_3$ [Jordanian code 4/5/4 page 73]

$S_1 = 1$ [Jordanian code No. 13]

$S_2 = 0.74$ [Jordanian code No. 14]

$S_3 = 1$ [Jordanian code No. 15]

$V_Z = 35 \times 1 \times 0.74 \times 1 = 25.9$ m/s

Dynamic wind pressure:-

$q = 0.613\,(V_Z)^2$ [Jordanian eq. 6]

$q = 0.613\,(25.9)^2 = 411.2$ N/m^2

So » 0.4 KN/m^2 will be.

0.4 KN/m$^2 \times 1 = 0.4$ KN/m

Wind strike the shed from the right and by analyzing the wind force on the slab a trivial value appears so we neglected.

Wind load on the slab = $0.4 \times \sin 15 = 0.1$ KN/m

This is very small value it will be **neglected.**

4.1.1.2 Load combination at the Slab

According to Aci, the following load combination is applied:-

Case 1:

$U = 1.2\ D + 1.6\ L$

$U = 1.2 \times 3.5 + 1.6 \times 1 = 5.8$ KN/m

Case 2:

$U = 1.2\ D + 1.6\ L + 0.5\ S$

$U = 1.2 \times 3.5 + 1.6 \times 1 + 0.5 \times 10 = 10.8$ KN/m

Case 3:

$U = 1.2\ D + 1.6\ S + 0.5\ L$

$U = 1.2 \times 3.5 + 1.6 \times 10 + 0.5 \times 1 = 20.7$ KN/m

Case 4:

$U = 1.2\ D + 1.6\ S + (\ L$ or $0.8\ W\ )$

$U = 1.2 \times 3.5 + 1.6 \times 10 + 1 = 21.2$ KN/m

Case 4: Gives the largest load and so the largest moment and shear forces.

4.1.1.3 Moment and Shear Envelop diagrams

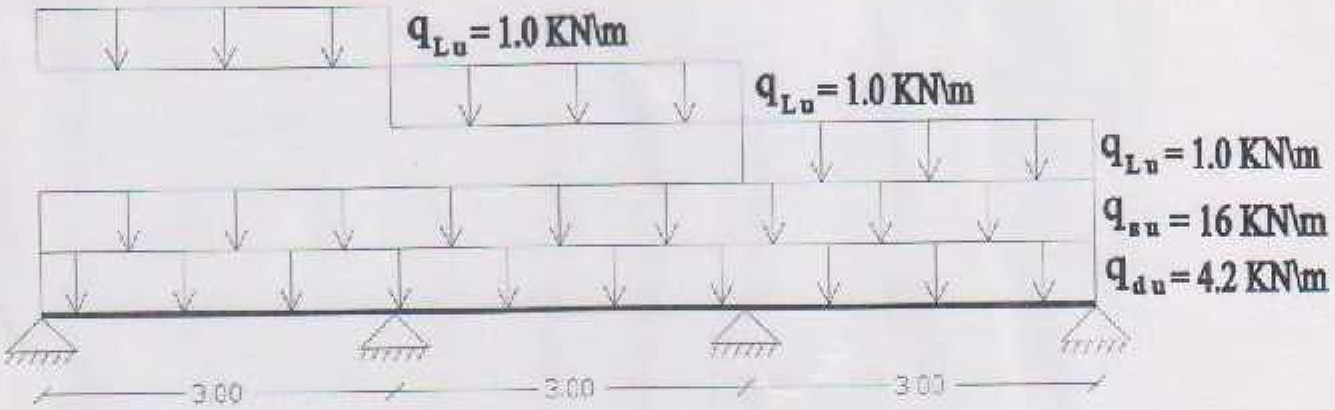


Figure (4-3): System of slab of parking shed with loads

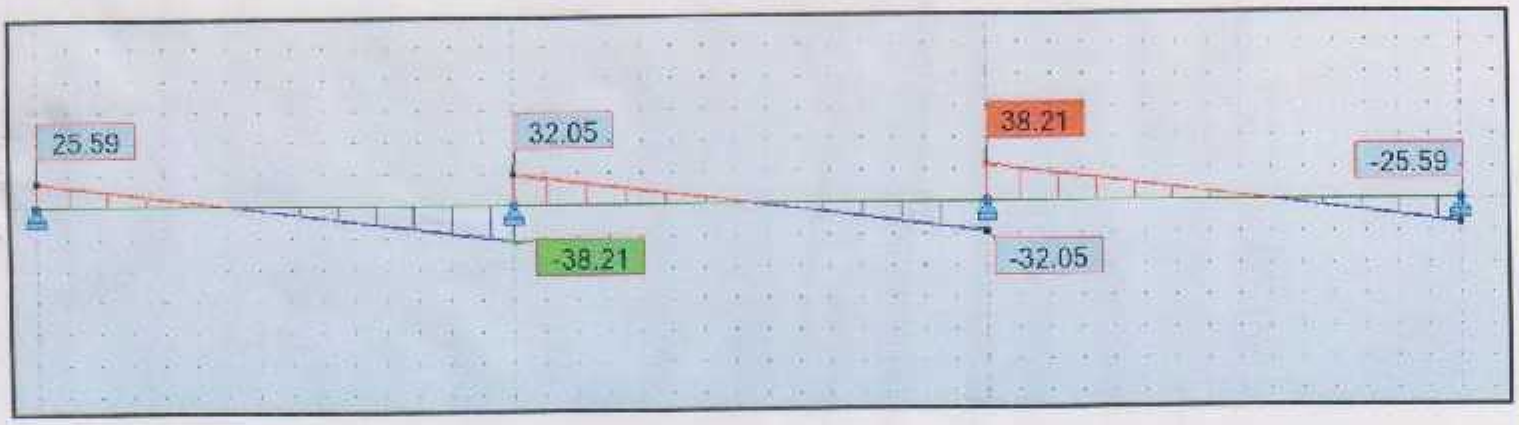


Figure (4-4): Slab Shear Envelop Diagram

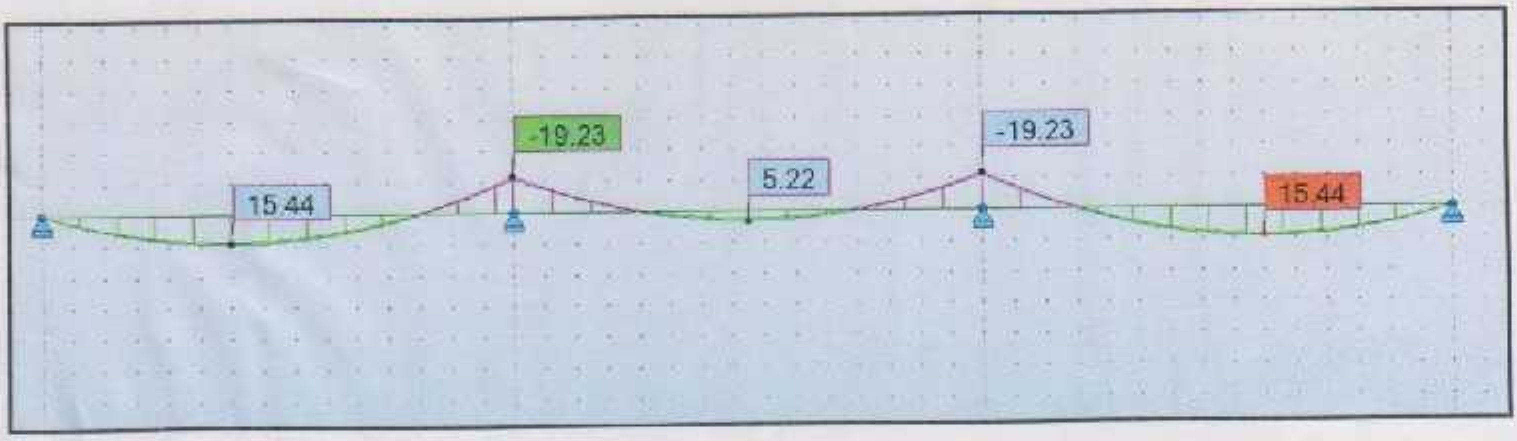


Figure (4-5): Slab Moment Envelop Diagram

4.1.1.4 Design of Shear

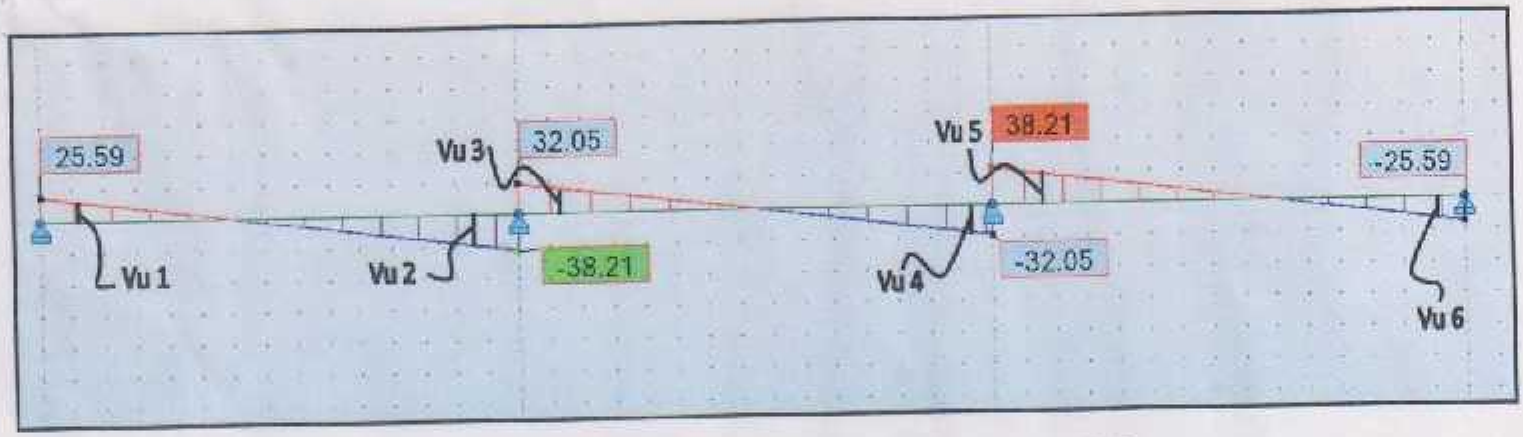


Figure (4-6): Shear force at the critical section – Slab

Vu at a distance d from face the face of support.

d = 140 - 25 - 6 = 109 mm

Vu at a distance = 250 + 109 = 359 mm from center of support.

$Vu_1 = Vu_6 = 25.59 - [0.359 \times 21.2] = 17.98$ KN

$Vu_2 = Vu_5 = 38.21 - [0.359 \times 21.2] = 30.60$ KN

$Vu_3 = Vu_4 = 32.05 - [0.359 \times 21.2] = 24.44$ KN

Check whether the thickness of the slab is adequate for shear:

Vu max = 30.60 KN.

$Vc = \frac{1}{6}\sqrt{f'_c}b_w d = \frac{1}{6}\sqrt{28} \times 1000 \times 109 \times 10^{-3} = 96 \frac{KN}{1m\ strip}$, ACI (11-3)

$\emptyset\ Vc = 0.75 \times 96 = 72 \frac{KN}{1m\ strip}$

Vu max= 30.60 KN < ∅ Vc = 72 KN

→No need to increase the slab thickness, its adequate enough.

→No shear reinforcement required.

4.1.1.5 Design of Bending Moment

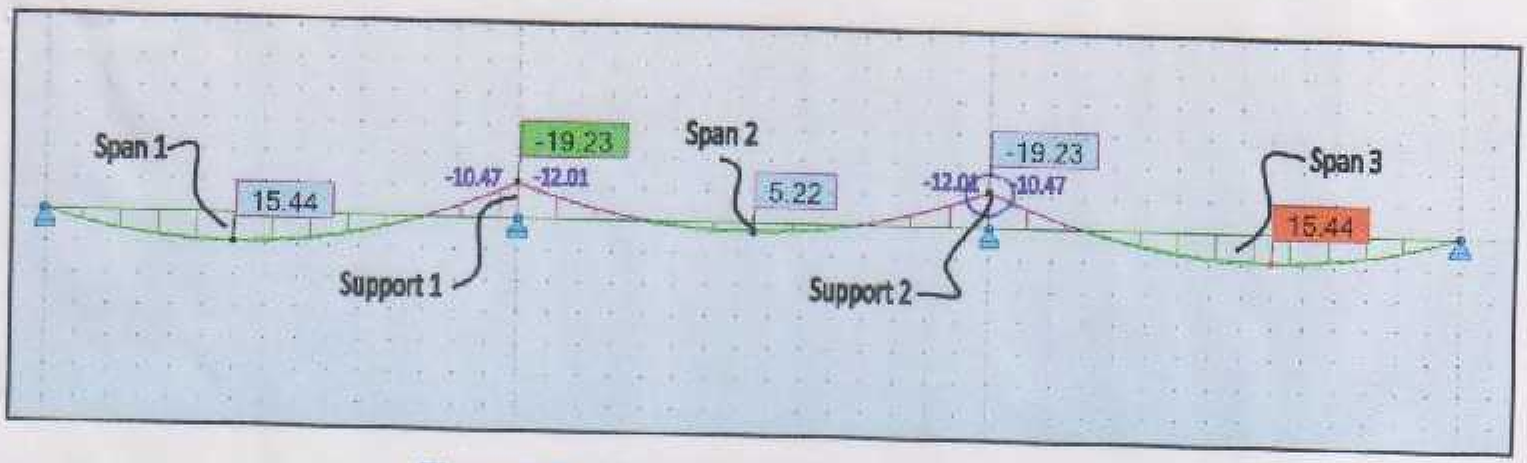


Figure (4-7): Moment at the face of supports – Slab

Design for positive moment:

Design of span (1) and (3)

$M_u = 15.44$ KN.m

$$Kn = \frac{M_u/\emptyset}{bd^2} = \frac{15.44 * 10^6/0.9}{1000 * (109)^2} = 1.44 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{414}{0.85 * 28} = 17.4$$

$$d = 140 - 25 - 6 = 109 \text{ mm}$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{17.4}\left(1 - \sqrt{1 - \frac{2 * 17.4 * 1.44}{414}}\right) = 0.0036$$

$As_{req} = \rho$ b d $= 0.0036 * 1000 * 109 = 392.4 \text{ mm}^2$

$As_{min} = 0.0018$ b h $= 0.0018 * 1000 * 140 = 252 \text{ mm}^2$, ACI-318-08 (7.12.2.1)

$As_{req} = 392.4 \text{ mm}^2 > As_{min} = 252 \text{ mm}^2$

Select Ø10/20cm , with $As = (\pi \times 10^2 / 4) \times 100/20 = 392.7 \text{ mm}^2$

$S \geq 25$ mm
$\geq \emptyset = 10$ mm

$S \leq 3 * h \rightarrow S \leq 3 * 14 \rightarrow S \leq 42$ cm

≤ 45 cm

$S_{max} = 42 \text{ cm} > S = 20 \text{ cm} > S_{min} = 2.5 \text{ cm}$

Check: strain

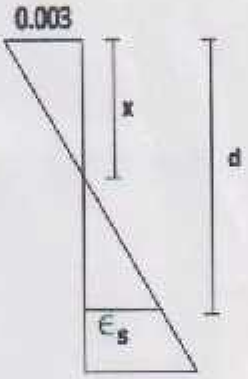
$T = C$

$As\, f_y = 0.85\, f_c'\, a\, b$

$392.7 * 414 = 0.85 * 28 * a * 1000$

$a = 6.83$ mm

$x = \dfrac{6.83}{0.85} = 8.04$ mm



$\dfrac{0.003}{x} = \dfrac{0.003 + \epsilon_s}{d}$

$\dfrac{0.003}{8.04} = \dfrac{0.003 + \epsilon_s}{109}$

$\epsilon_s = 0.038 > 0.005$

OK $\rightarrow \emptyset = 0.9$

Design of span (2)

$M_u = 5.22$ KN.m

$Kn = \dfrac{M_u/\emptyset}{bd^2} = \dfrac{5.22 * 10^6/0.9}{1000 * (109)^2} = 0.49$ Mpa

$m = \dfrac{f_y}{0.85 f_c'} = \dfrac{414}{0.85 * 28} = 17.4$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{17.4}\left(1 - \sqrt{1 - \frac{2 * 17.4 * 0.49}{414}}\right) = 0.0012$$

$As_{req} = \rho\, b\, d = 0.0012 * 1000 * 109 = 130.8 \text{ mm}^2$

$As_{min} = 0.0018\, b\, h = 0.0018 * 1000 * 140 = 252 \text{ mm}^2$

$As_{req} = 130.8 \text{ mm}^2 < As_{min} = 252 \text{ mm}^2$

$As_{req} = 252\ mm^2$

Select Ø10/30cm, with $As = (\pi \times 10^2 / 4) \times 100/30 = 261.8\ mm^2$

$S \geq 25$ mm
$\geq Ø = 10$ mm

$S \leq 3 * h \rightarrow S \leq 3 * 14 \rightarrow S \leq 42$ cm
≤ 45 cm

$S_{max} = 42\ cm > S = 30\ cm > S_{min} = 2.5\ cm$

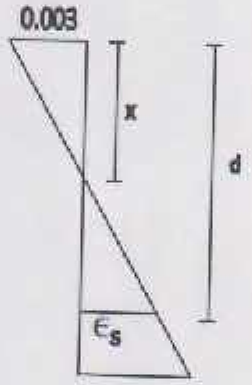
Check: strain

$T = C$

$As\, f_y = 0.85\, f_c'\, a\, b$

$a = 4.55$ mm

$x = 5.36$ mm



$$\frac{0.003}{x} = \frac{0.003+\epsilon_s}{d}$$

$$\frac{0.003}{5.36} = \frac{0.003+\epsilon_s}{109}$$

$\epsilon_s = 0.058 > 0.005$

OK $\rightarrow Ø = 0.9$

Design for negative moment:

Design of support (1) and (2)

$M_u = -12.01$ KN.m

$$Kn = \frac{M_u/Ø}{bd^2} = \frac{12.01 * 10^6/0.9}{1000 * (109)^2} = 1.123\ Mpa$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{414}{0.85 * 28} = 17.4$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{17.4}\left(1 - \sqrt{1 - \frac{2 * 17.4 * 1.123}{414}}\right) = 0.0028$$

$As_{req} = \rho\, b\, d = 0.0028 * 1000 * 109 = 305.2\ mm^2$

$As_{min} = 0.0018\, b\, h = 0.0018 * 1000 * 140 = 252\ mm^2$

$As_{req} = 305.2\ mm^2 > As_{min} = 252\ mm^2$

Select Ø10/25cm , with $As = (\pi \times 10^2 / 4) \times 100/25 = 314.16\ mm^2$

$S \geq 25$ mm

$\geq Ø = 12$ mm

$S \leq 3 * h \rightarrow S \leq 3 * 14 \rightarrow S \leq 42$ cm

≤ 45 cm

$S_{max} = 42\ cm > S = 25\ cm > S_{min} = 2.5\ cm$

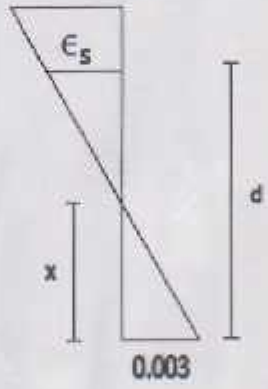
Check: strain

$T = C$

$As\, f_y = 0.85\, f_c^{'}\, a\, b$

$a = 5.46$ mm

$x = 6.43$ mm



$$\frac{0.003}{x} = \frac{0.003 + \epsilon_s}{d}$$

$$\frac{0.003}{6.43} = \frac{0.003 + \epsilon_s}{109}$$

$\epsilon_s = 0.048 > 0.005$

So $\rightarrow Ø = 0.9$

In the secondary direction to cover the secondary moment, shrinkage and temperature is required:

$As_{min} = 0.0018\, b\, h = 0.0018 * 1000 * 140 = 252\ mm^2$

Select Ø10/30cm , with $As = (\pi \times 10^2 / 4) \times 100/30 = 261.8\ mm^2$

4.1.2 Design of Intermediate Frame

4.1.2.1 Calculation of loads

Dead load of slab

$0.14 \times 25 \times 3 = 10.5$ KN/m

Weight of the drop part of the beam

$(0.66 + 0.06) / 2) \times 0.5 \times 25 = 4.5$ KN/m

Total dead load from slab = 4.5 + 10.5 = 15 KN/m

Live Load

[1 KN/m^2] will be applied.

$1 KN/m^2 \times 3 = 3$ KN/m

Snow load

Because of the risk of Accumulation of the snow on the frame beam of the shed a depth of 1m of ice will be considered:

[According to Jordanian code 2/8/3 page 62]

$1 \times [10\ KN/m^3] \times 3 = 30$ KN/m

Wind load

According to Jordanian code the wind load will be calculated as follows:

Basic wind speed V = 35 m/s [assumed]

Design speed:-

$V_Z = V \times S_1 \times S_2 \times S_3$ [Jordanian code 4/5/4 page 73]

$S_1 = 1$ [Jordanian code No. 13]

$S_2 = 0.74$ [Jordanian code No. 14]

$S_3 = 1$ [Jordanian code No. 15]

$V_Z = 35 \times 1 \times 0.74 \times 1 = 25.9$ m/s

Dynamic wind pressure:-

$q = 0.613\ (V_Z)^2$ [Jordanian eq. 6]

$q = 0.613\ (25.9)^2 = 411.2\ N/m^2$

$q = 0.4\ KN/m^2$ will be applied.

$0.4\ KN/m^2 \times 3 = 1.2$ KN/m at the frame column.

Wind strike the shed from the right and by analyzing the wind force on the frame beam a trivial value appears so we neglected.

Wind load on the frame beam = $0.4 \times \sin 15 = 0.1$ KN/m^2

This is very small value it will be **neglected.**

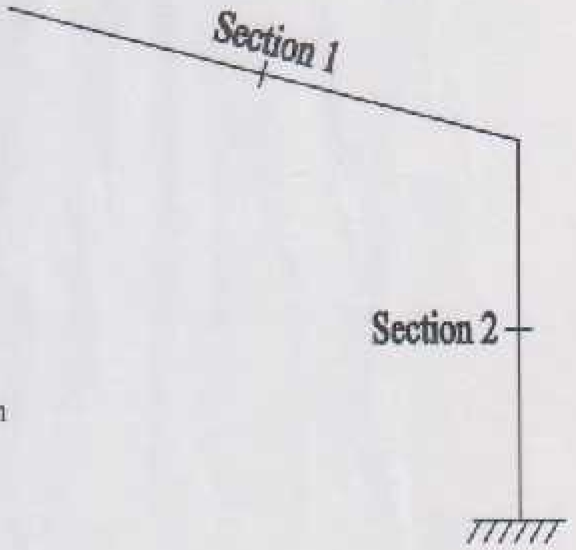
Weight of column = $0.8 \times 0.5 \times 25 \times 3.5 = 35$ KN (point load)

4.1.2.2 Load combination at the Frame

Load combination at the intermediate frame (Section1):-

According to Aci, the following load combination is applied:-

Case 1:



$U = 1.2\ D + 1.6\ L$

$U = 1.2 \times 15 + 1.6 \times 3 = 22.8$ KN/m

Case 2:

$U = 1.2\ D + 1.6\ L + 0.5\ S$

$U = 1.2 \times 15 + 1.6 \times 3 + 0.5 \times 30 = 37.8$ KN/m

Case 3:

$U = 1.2\ D + 1.6\ S + 0.5\ L$

$U = 1.2 \times 15 + 1.6 \times 30 + 0.5 \times 3 = 37.5$ KN/m

Case 4:

U = 1.2 D + 1.6 S + (L or 0.8 W)

$U = 1.2 \times 15 + 1.6 \times 30 + 3 = 69$ KN/m

Case 4: Gives the largest load and so the largest moment and shear forces.

Load combination at the Intermediate frame (Section2):-

According to Aci, the following load combination is applied:-

Dead = 1.2 D = 1.2 × 35 = 42 KN

Wind = 1.6 W = 1.6 × 1.2 = 1.92 KN/m

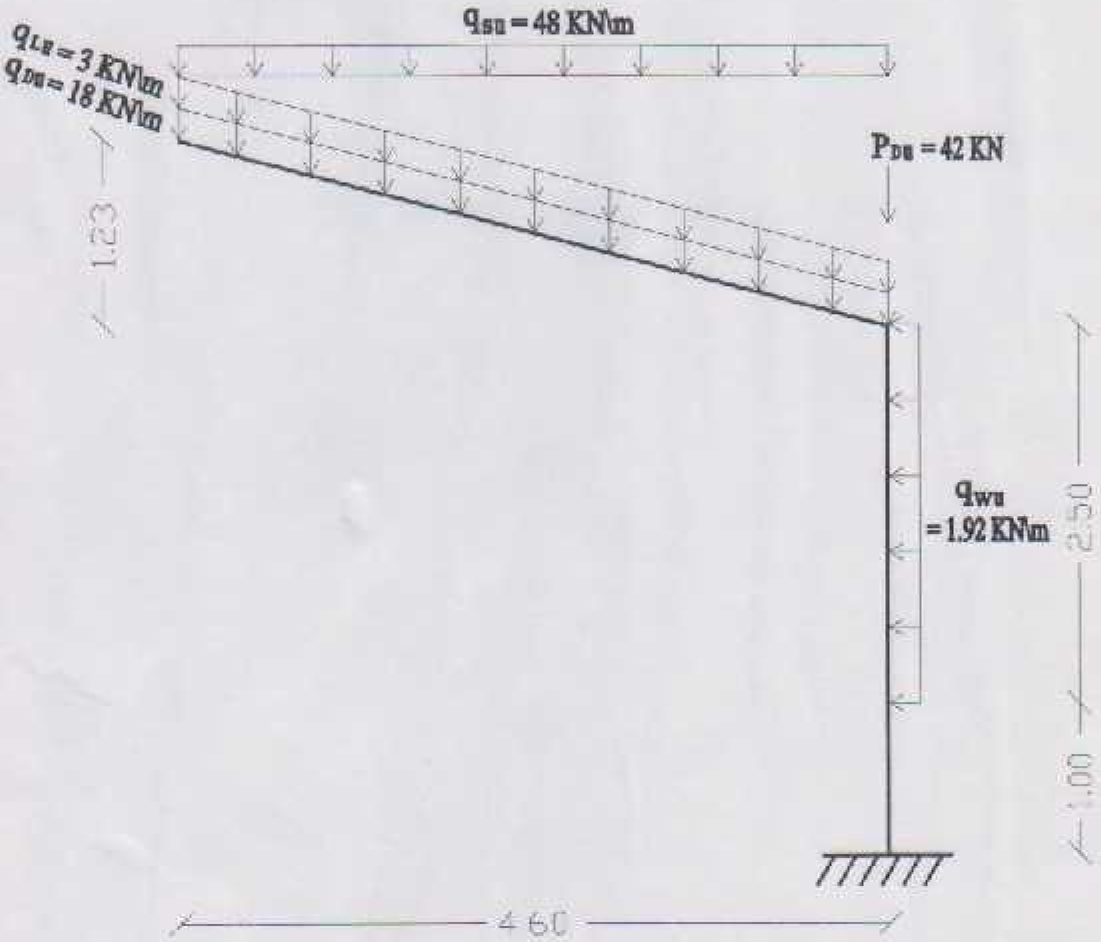


Figure (4-8): System of Intermediate Frame

4.1.2.3 Internal forces diagrams

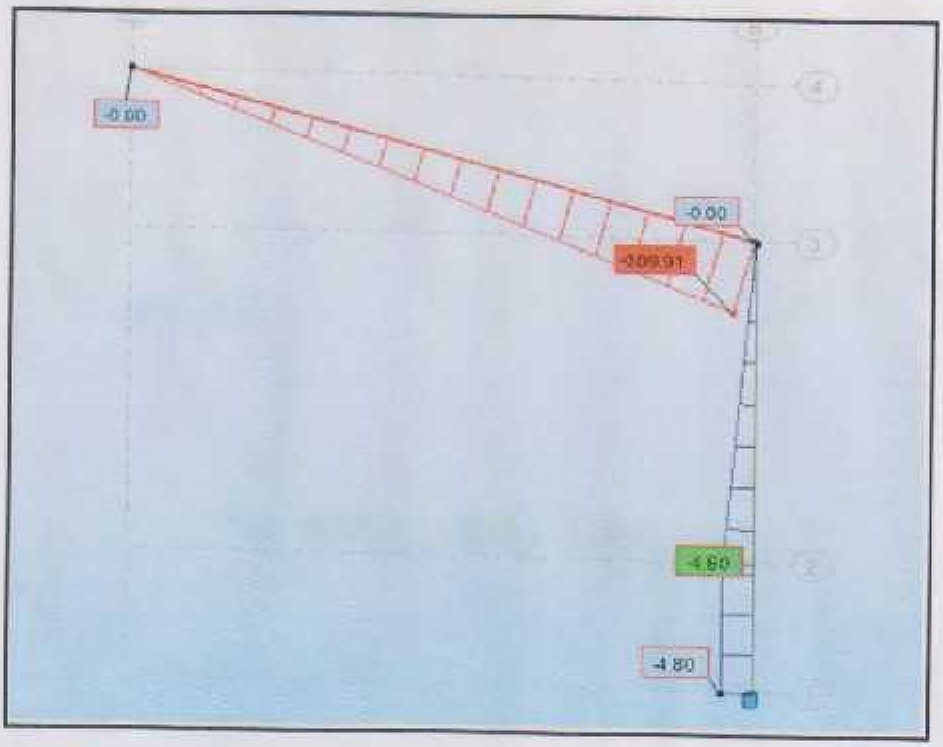


Figure (4-9): Intermediate Frame Shear Force Diagram

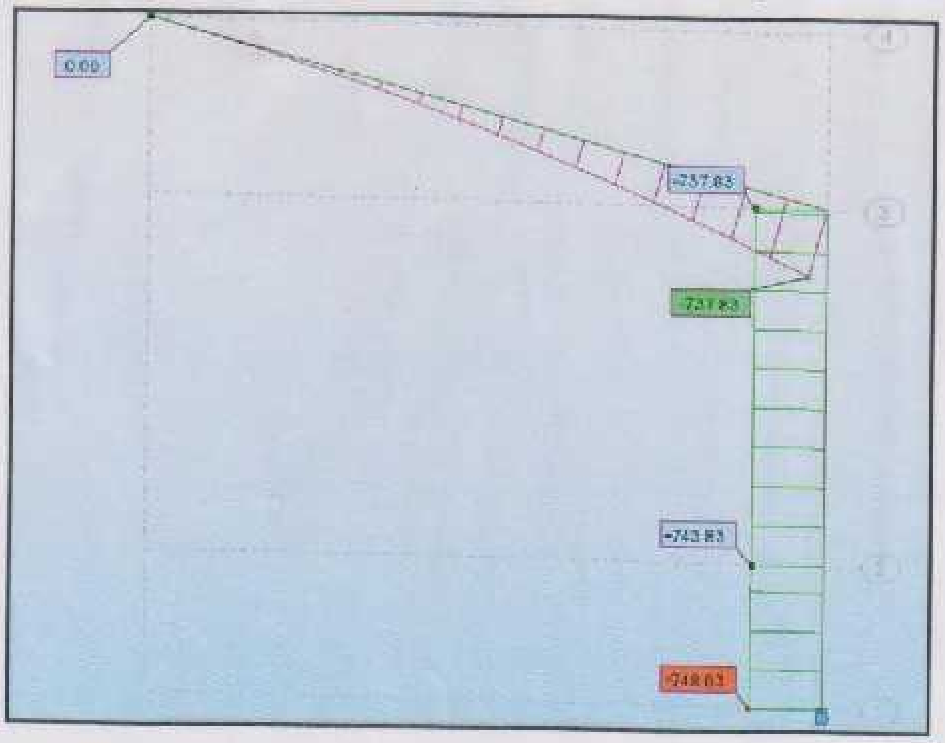


Figure (4-10): Intermediate Frame Moment Diagram

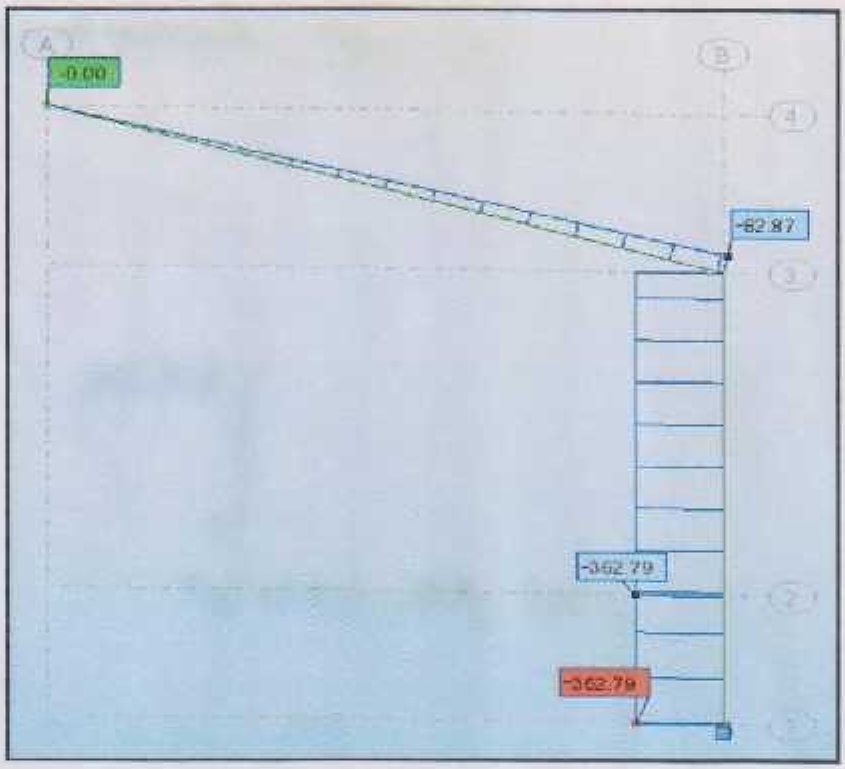


Figure (4-11): Intermediate Frame Normal Force diagram

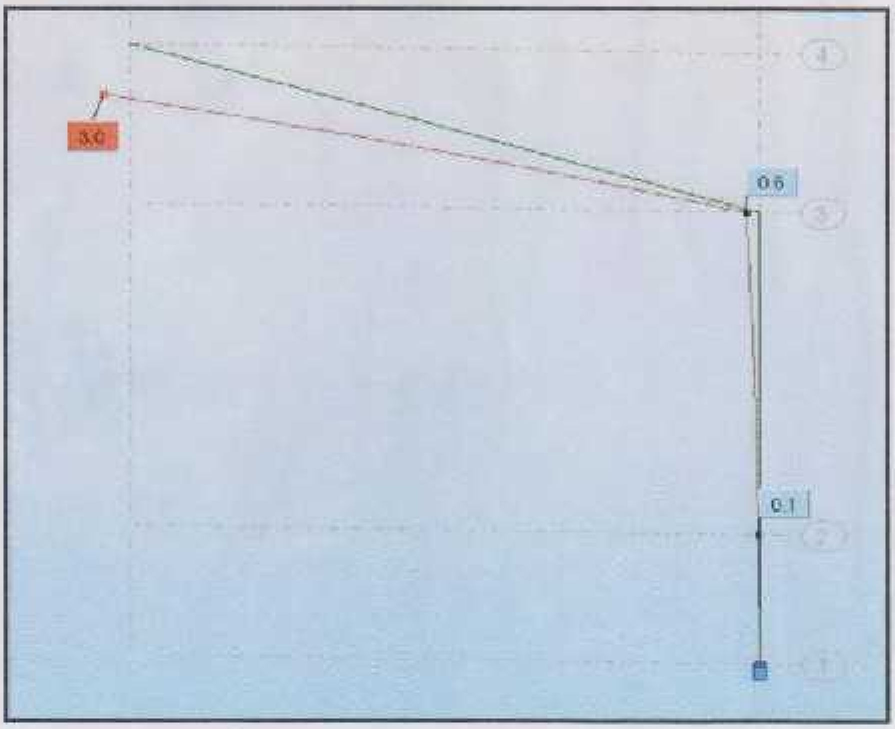


Figure (4-12): Intermediate Frame Deformation diagram

Check Deflection:

Allowable deflection = L / 125 = 460 / 125 = 3.7 cm

Max deflection = 3 cm < 3.7 cm Ok

4.1.2.4 Design of Intermediate frame beam

4.1.2.4.1 Design of Bending Moment

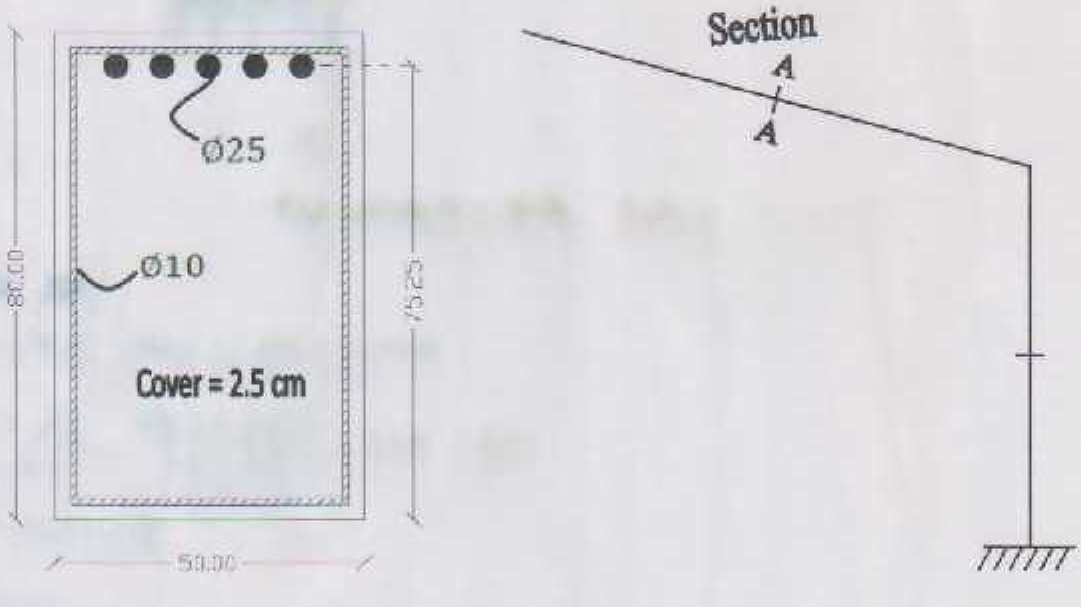


Figure (4-13): Beam section (A-A) for Intermediate Frame

M_u at the face = - 619.45 KN.m

$$d = 800 - 25 - 10 - (\frac{25}{2}) = 752.5 \text{ mm}$$

$$Kn = \frac{M_u/\emptyset}{bd^2} = \frac{619.45 * 10^6/0.9}{500 * (752.5)^2} = 2.43 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{414}{0.85 * 28} = 17.4$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{17.4}\left(1 - \sqrt{1 - \frac{2 * 17.4 * 2.43}{414}}\right) = 0.0062$$

$As_{req} = \rho\, b\, d = 0.0062 * 500 * 752.5 = 2332.75\ mm^2$

$As_{min} = 0.25 * \frac{\sqrt{f_c'}}{f_y} * b * d = 0.25 * \frac{\sqrt{28}}{414} * 500 * 752.5 = 1202.25\ mm^2$

Not less than

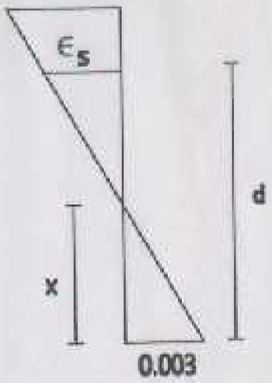
$As_{min} = \frac{1.4}{f_y} * b * d = \frac{1.4}{414} * 500 * 752.5 = 1272.34\ mm^2$, ACI-318-08 (10.5.1)

$As_{req} = 2332.75\ mm^2 > As_{min} = 1272.34\ mm^2$

$n = \frac{2332.75}{491} = 4.75$

Select 5Ø25

Check: strain



$T = C$

$As\, f_y = 0.85\, f_c'\, a\, b$

$5 * 491 * 414 = 0.85 * 28 * a * 500$

$a = 85.41\ mm$

$x = \frac{85.41}{0.85} = 100.48\ mm$

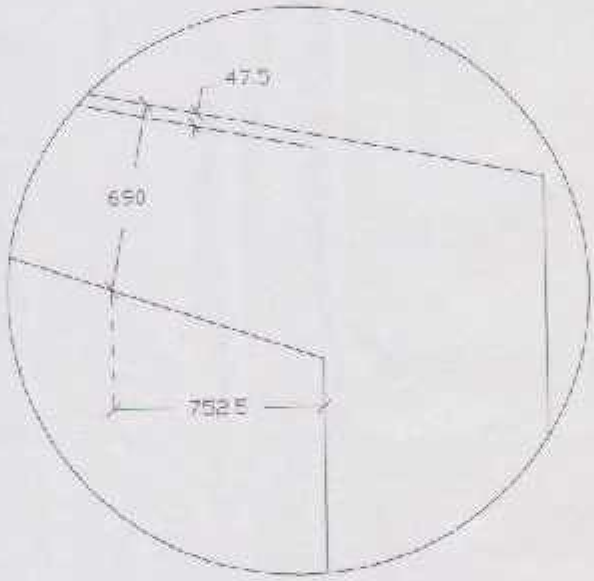
$\frac{0.003}{x} = \frac{0.003 + \epsilon_s}{d}$

$\frac{0.003}{100.48} = \frac{0.003 + \epsilon_s}{752.5}$

$\epsilon_s = 0.019 > 0.005$

So → $\emptyset = 0.9$

4.1.2.4.2 Design of Shear



Critical section at 752.5 from face of column

Vu = - 228.8 KN

Nu = - 61.18 KN

d at the critical section = 690 – 47.5 = 642.5 mm

$$V_c = 0.17 * \left(1 + \frac{Nu}{14\ Ag}\right) * \sqrt{f_c'} * b_w * d \qquad ACI\ (11-4)$$

$$V_c = 0.17 * \left(1 + \frac{61.18 * 10^3}{14 * 500 * 690}\right) * \sqrt{28} * 500 * 642.5 = 292.64\ KN \ \text{contol}$$

Vc not greater than:

$$V_c = 0.29 * \sqrt{f_c'} * b_w * d * \sqrt{1 + \frac{0.29 * Nu}{Ag}} \qquad ACI\ (11-7)$$

$$V_c = 0.29 * \sqrt{28} * 500 * 642.5 * \sqrt{1 + \frac{0.29 * 61.18 * 10^3}{500 * 690}} = 505.49\ KN$$

<u>Region 3</u>

$V_u \leq \emptyset\ V_c + \emptyset\ V_{s\,min}$

$\emptyset\ V_{s\,min} = 0.75\ \frac{1}{16}\sqrt{f_c'}\ b_w\ d = 0.75 * \frac{1}{16} * \sqrt{28} * 500 * 642.5 = 79.68\ KN$

$\emptyset\ V_{s\,min} = 0.75\ \frac{1}{3}\ b_w\ d = 0.75 * \frac{1}{3} * 500 * 642.5 = 80.31\ KN$ Controls

$\emptyset\ V_c = 0.75 * 292.64 = 219.48\ KN$

$V_u = 228.8 \leq 219.48 + 80.31$

Minimum shear reinforcement is required

Use two legs with $Av = 2 \times (\pi \times 10^2 / 4) = 157\ mm^2$

$$S = \frac{Av * f_y * d}{V_{s\,min}} = \frac{157 * 414 * 642.5}{\frac{80.31}{0.75} * 10^3} = 390\ mm$$

$$S = \frac{642.5}{2} = 321.25\ mm$$

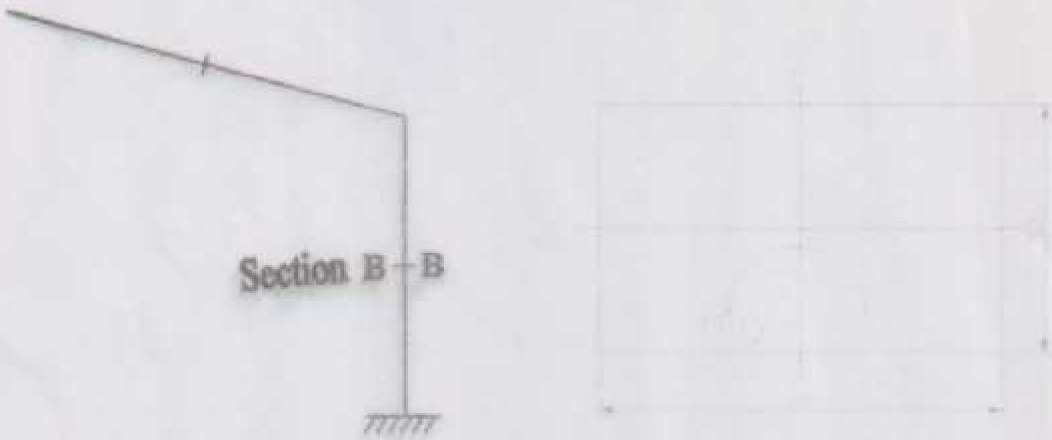
$S \leq d/2$

$S \leq 600\ mm$

Select Ø10/30 cm

4.1.2.5 Design of Intermediate frame column

4.1.2.5 Design of Intermediate frame column



Column section (B-B) for Intermediate Frame

Stability index:

$\Delta_\circ$= Laterl deflection for ultimate case

$$Q = \frac{Pu \times \Delta_\circ}{V_u \times L_c} = \frac{362.8 \times 0.009}{4.8 \times 3.5} = 0.194 > 0.05$$

→Unbraced column

Slenderness:

$$\frac{KL}{r} \le 22$$

System about x-axis

$$\frac{KL}{r} \le 22 = \frac{2 \times 280}{0.3 \times 50} = 37.3 > 22 \rightarrow \text{Long Column}$$

System about y-axis

$$\frac{KL}{r} \le 22 = \frac{2 \times 280}{0.3 \times 80} = 23.3 > 22 \rightarrow \text{Long Column}$$

Bresler equation:

$$\frac{1}{P_n} = \frac{1}{P_{nx}}$$

$$P_n = P_{nx}$$

System about y-axis (in x -direction)

$$e = \frac{mu}{p_u} = \frac{748.63}{362.79} = 2.06 \text{ m}$$

$$\frac{e}{h} = \frac{2.06}{0.8} = 2.58$$

$$\frac{\gamma}{h} = \frac{800 - 50 - 20 - 25}{800} = 0.88$$

$$\frac{\emptyset \times p_n}{A_g} = \frac{p_u}{A_g} = \frac{362.79/1000}{(0.5 \times 0.8)} = 0.907 \text{ mpa} = 0.132 \text{ ksi}$$

$$\frac{\emptyset \times m_n}{A_g \times h} = \frac{m_u}{A_g \times h} = \frac{748.63/1000}{(0.5 \times 0.8 \times 0.8)} = 2.34 \text{ mpa} = 0.34 \text{ ksi}$$

Simulate these values on the interaction diagram:

For $\frac{\gamma}{h} = 0.75 \rightarrow \rho_g = 0.0138$

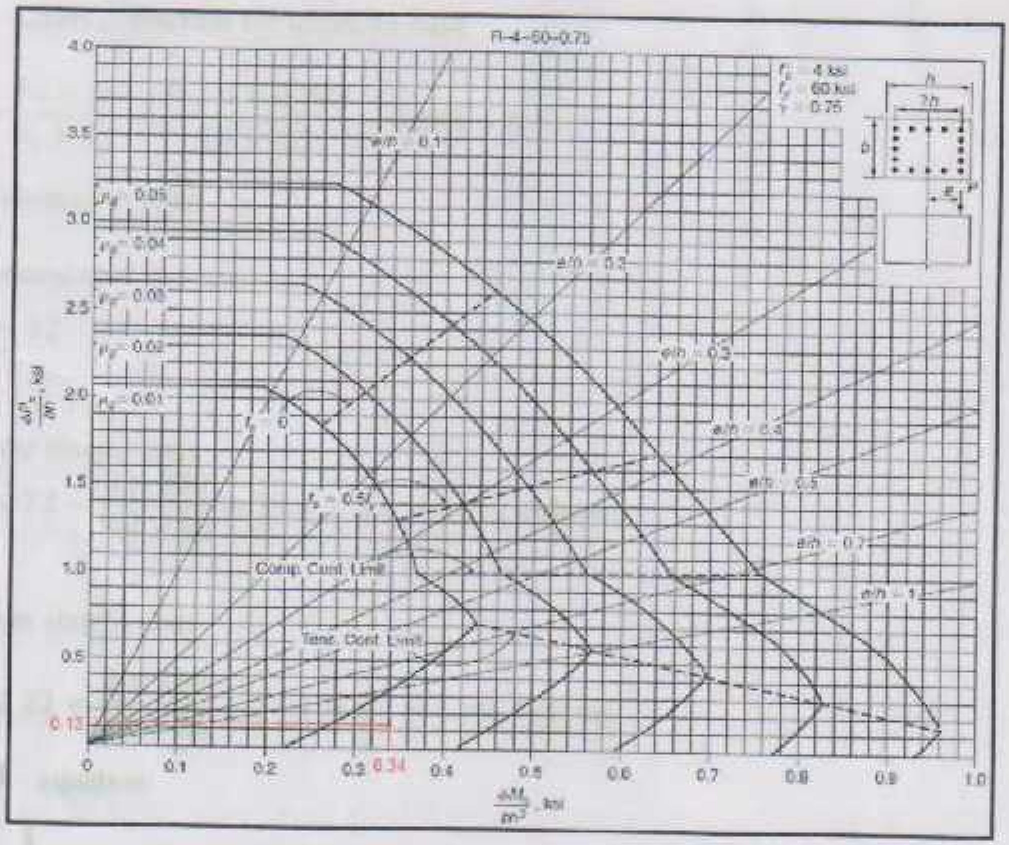


Figure (4-15): Interaction Diagram (R - 4 - 60 - 0.75)

For $\frac{\gamma}{h} = 0.90 \rightarrow \rho_g = 0.012$

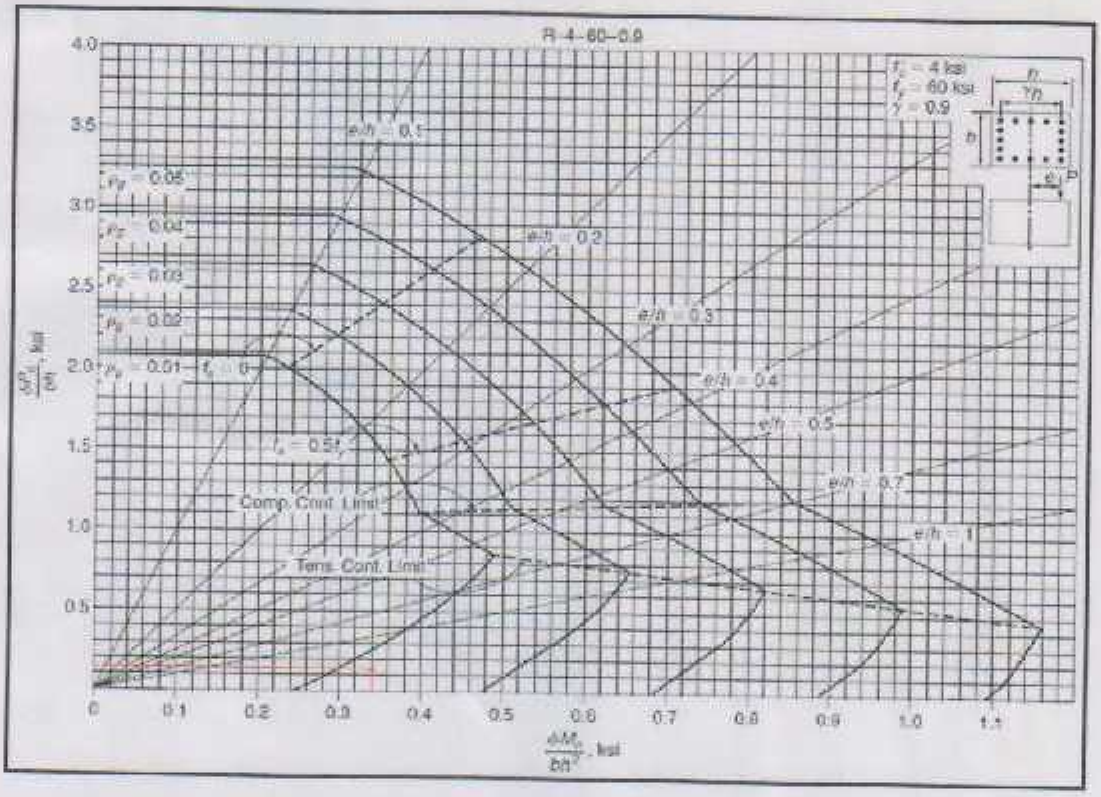


Figure (4-16): Interaction Diagram (R - 4 - 60 - 0.90)

By interpolation ρ_g at $\frac{\gamma}{h} = 0.88 \rightarrow \rho_g = 0.0122$

$As_{req} = \rho_g \times A_g = 0.0122 \times 500 \times 800 = 4880\ mm^2$

$n = \frac{4880}{491} = 9.9$

Select 10Ø25

We try to design the column as flexural member and the result was 6Ø25 ,which is less than 10Ø25.

Design of stirrups:

Construction rules from ACI for tied reinforced section:

For diameter of ties:

For $\emptyset_{\text{Vertical bars}} = 25$ mm → Select $\emptyset_{\text{Stirrups}} = 10$ mm

Spacing of ties:

$S \leq 16 \times \emptyset_{\text{vertical bars}} = 16 \times 25 = 400$ mm

$S \leq 48 \times \emptyset_{\text{stirrups}} = 48 \times 10 = 480$ mm

$S \leq$ minimum dimenstion if colummn = 500 mm

$S = 400$ mm → **control**.

4.1.3 Design of Exterior Frame

4.1.3.1 Calculation of loads

Dead load of slab
$0.14 \times 25 \times 1.5 = 5.25$ KN/m

Weight of the drop part of the beam
$(0.66 + 0.06) / 2) \times 0.5 \times 25 = 4.5$ KN/m

Total dead load from slab = $4.5 + 5.25 = 9.75$ KN/m

Live Load
[1 KN/m^2] will be applied.
1KN/m^2 $\times$ 1.5 = 1.5 KN/m

Snow load
Because of the risk of Accumulation of the snow on the frame beam of the shed a depth of 1m of ice will be considered:

[According to Jordanian code 2/8/3 page 62]
1$\times$ [10 KN/m^3] $\times$ 1.5 = 15 KN/m

Wind load

According to Jordanian code the wind load will be calculated as follows:

Basic wind speed V = 35 m/s [assumed]

Design speed:-

$V_Z = V \times S_1 \times S_2 \times S_3$ [Jordanian code 4/5/4 page 73]

$S_1 = 1$ [Jordanian code No. 13]

$S_2 = 0.74$ [Jordanian code No. 14]

$S_3 = 1$ [Jordanian code No. 15]

$V_Z = 35 \times 1 \times 0.74 \times 1 = 25.9$ m/s

Dynamic wind pressure:-

$q = 0.613\,(V_Z)^2$ [Jordanian eq. 6]

$q = 0.613\,(25.9)^2 = 411.2\ N/m^2$

$q = 0.4\ KN/m^2$ will be applied.

$0.4\ KN/m^2 \times 1.5 = 0.6$ KN/m at the frame column.

Wind load on the frame beam = $0.4 \times \sin 15 = 0.1\ KN/m^2$

Wind strike the shed from the right and by analyzing the wind force on the frame beam a trivial value appears so we neglected.

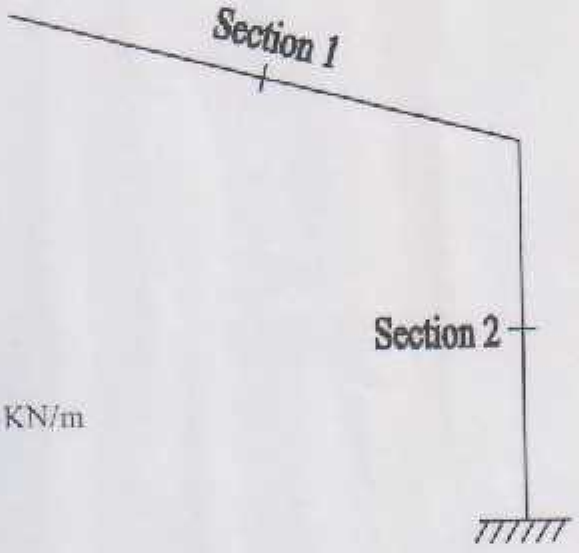
This is very small value it will be **neglected.**

Weight of column = $0.8 \times 0.5 \times 25 \times 3.5 = 35$ KN (point load)

4.1.3.2 Load combination at the Frame

Load combination at the Exterior frame (Section1):-

According to Aci, the following load combination is applied:-



Case 1:

$U = 1.2\ D + 1.6\ L$

$U = 1.2 \times 9.75 + 1.6 \times 1.5 = 14.1$ KN/m

Case 2:

$U = 1.2\ D + 1.6\ L + 0.5\ S$

$U = 1.2 \times 9.75 + 1.6 \times 1.5 + 0.5 \times 15 = 21.6$ KN/m

Case 3:

$U = 1.2\ D + 1.6\ S + 0.5\ L$

$U = 1.2 \times 9.75 + 1.6 \times 15 + 0.5 \times 1.5 = 36.45$ KN/m

Case 4:

$U = 1.2\ D + 1.6\ S + (\ L$ <u>or</u> $0.8\ W\ )$

$U = 1.2 \times 9.75 + 1.6 \times 15 + 1.5 = 37.2$ KN/m

Case 4: Gives the largest load and so the largest moment and shear forces.

Load combination at the Exterior frame (Section2):-

According to Aci, the following load combination is applied:-

Dead $= 1.2\ D = 1.2 \times 35 = 42$ KN

Wind $= 1.6\ W = 1.6 \times 0.6 = 0.96$ KN/m

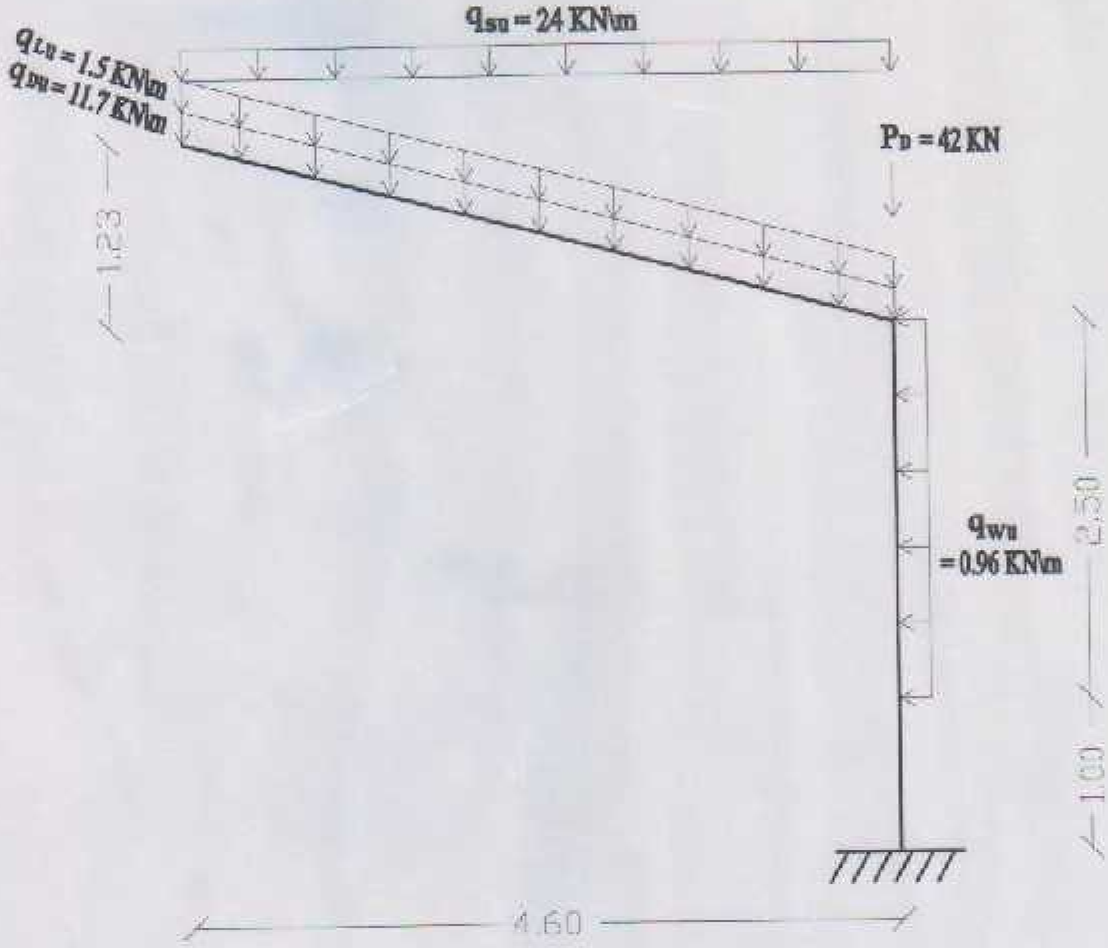


Figure (4-17): System of Exterior Frame

4.1.3.3 Internal forces diagrams

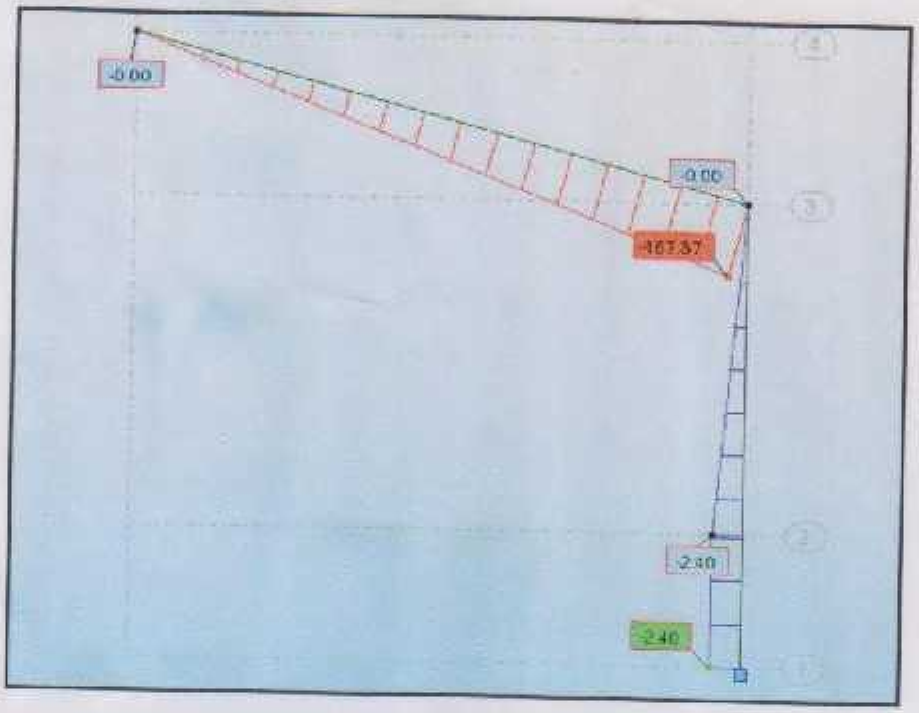


Figure (4-18): Exterior Frame Shear Force Diagram

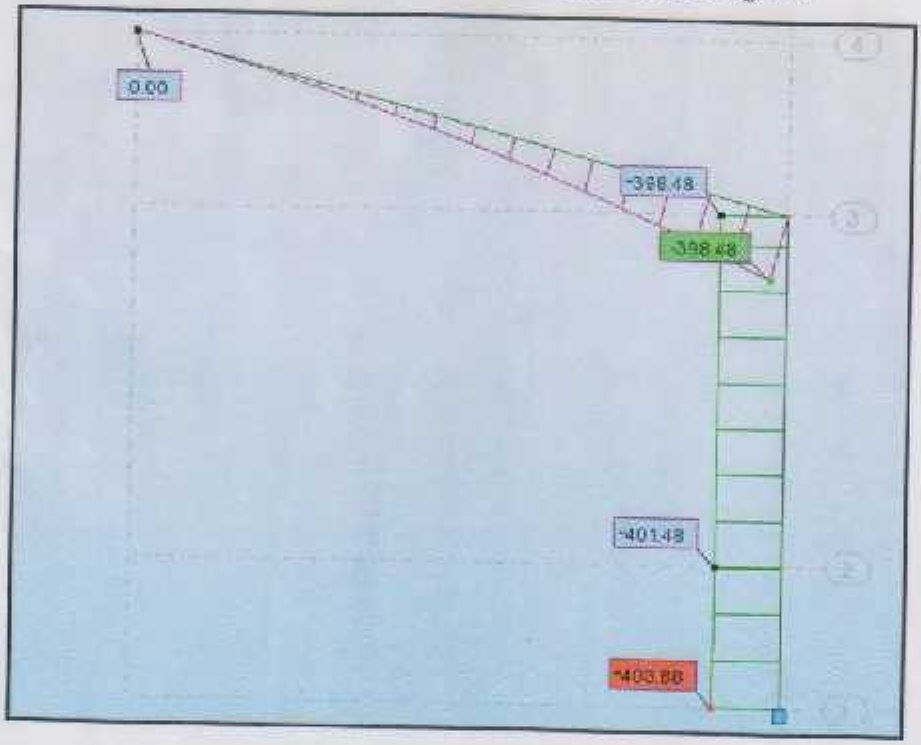


Figure (4-19): Exterior Frame Moment Diagram

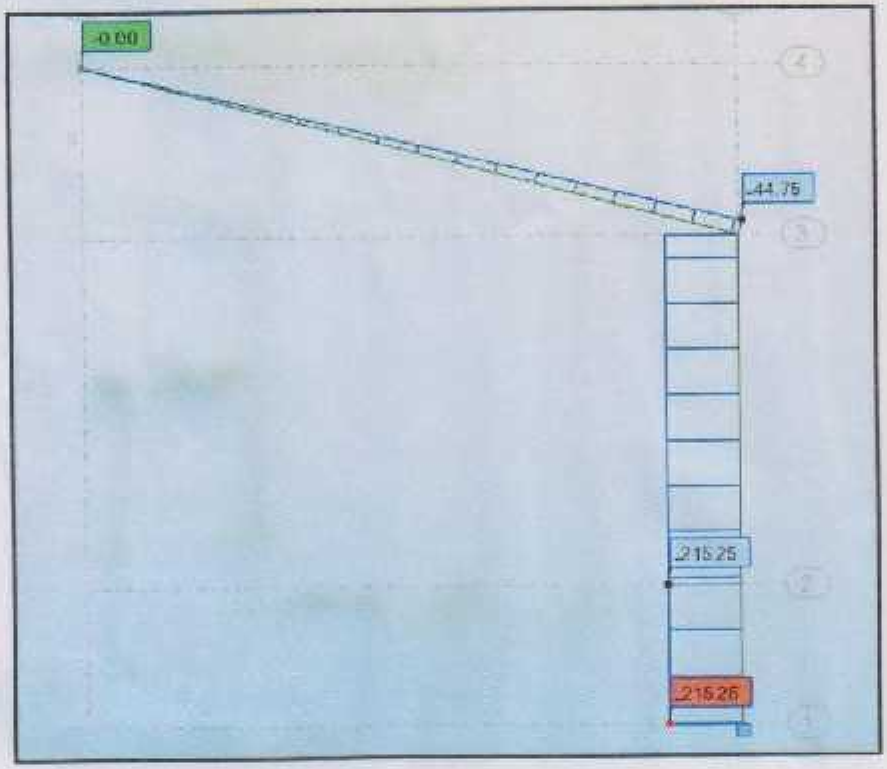


Figure (4-20): Exterior Frame Normal Force diagram

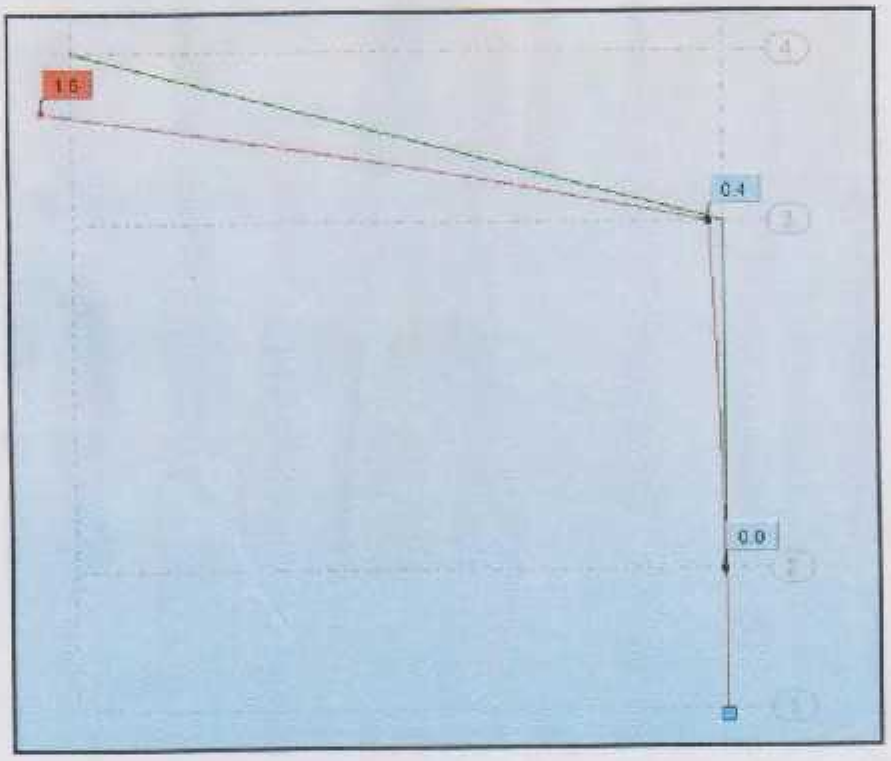


Figure (4-21): Exterior Frame Deformation diagram

Check Deflection:

Allowable deflection = L / 125 = 460 / 125 = 3.7 cm

Max deflection = 1.6 cm < 3.7 cm Ok

4.1.3.4 Design of Exterior frame beam

4.1.3.4.1 Design of Bending Moment

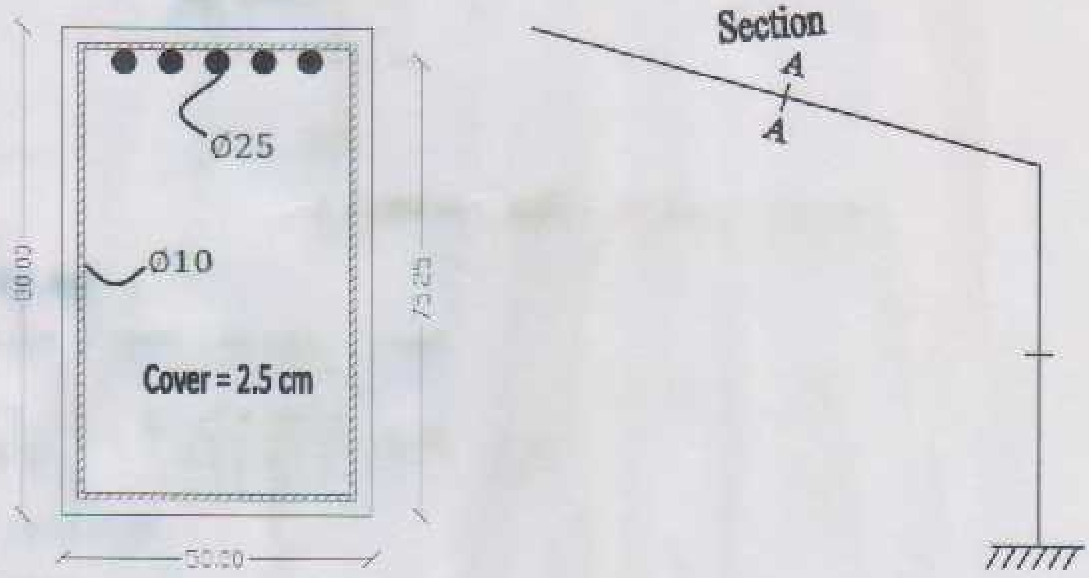


Figure (4-22): Beam section (A-A) for Exterior Frame

M_u at the face = - 332.01 KN.m

$$d = 800 - 25 - 10 - (\frac{25}{2}) = 752.5 \text{ mm}$$

$$Kn = \frac{M_u/\emptyset}{bd^2} = \frac{332.01 * 10^6/0.9}{500 * (752.5)^2} = 1.30 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{414}{0.85 * 28} = 17.4$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{17.4}\left(1 - \sqrt{1 - \frac{2 * 17.4 * 1.30}{414}}\right) = 0.0032$$

$$As_{req} = \rho \, b \, d = 0.0032 * 500 * 752.5 = 1204 \text{ mm}^2$$

$$As_{min} = 0.25 * \frac{\sqrt{f_c}}{f_y} * b * d = 0.25 * \frac{\sqrt{28}}{414} * 500 * 752.5 = 1202.25\ mm^2$$

Not less than

$$As_{min} = \frac{1.4}{f_y} * b * d = \frac{1.4}{414} * 500 * 752.5 = 1272.34\ mm^2$$

$$As_{req} = 1204\ mm^2 < As_{min} = 1272.34\ mm^2$$

$$As_{req} = 1272.34\ mm^2$$

$$n = \frac{1272.34}{254} = 5$$

Select 5Ø18

Check: strain

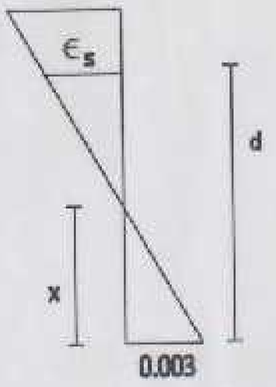
$T = C$

$As\ f_y = 0.85\ f_c\ a\ b$

$5 * 254 * 414 = 0.85 * 28 * a * 500$

$a = 44.18\ mm$

$$x = \frac{44.18}{0.85} = 51.98\ mm$$



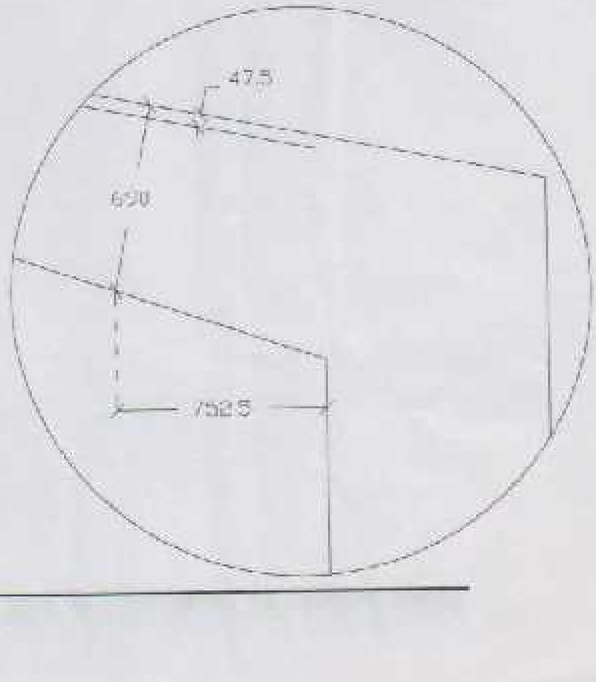
$$\frac{0.003}{x} = \frac{0.003+\epsilon_s}{d}$$

$$\frac{0.003}{51.98} = \frac{0.003+\epsilon_s}{752.5}$$

$\epsilon_s = 0.04 > 0.005$

So → $\emptyset = 0.9$

4.1.3.4.2 Design of Shear



Critical section at 752.5 from face of column

Vu = - 125.33 KN

Nu = - 33.5 KN

d at the critical section = 690 – 47.5 = 642.5 mm

$$V_c = 0.17 * \left(1 + \frac{Nu}{14\ Ag}\right) * \sqrt{f_c} * b_w * d \qquad ACI\ (11-4)$$

$$V_c = 0.17 * \left(1 + \frac{33.5 * 10^3}{14 * 500 * 690}\right) * \sqrt{28} * 500 * 642.5 = 290.98\ KN\ \text{contol}$$

Vc not greater than:

$$V_c = 0.29 * \sqrt{f_c'} * b_w * d * \sqrt{1 + \frac{0.29 * Nu}{Ag}} \quad ACI\ (11-7)$$

$$V_c = 0.29 * \sqrt{28} * 500 * 642.5 * \sqrt{1 + \frac{0.29 * 33.5 * 10^3}{500 * 690}} = 499.86\ KN$$

$\emptyset V_c = 0.75 * 290.98 = 218.24$ KN

$V_u = 125.33 \leq 218.24$

Minimum shear reinforcement is required

$V_{s\,min} = \frac{1}{16} \sqrt{f_c}\, b_w\, d = \frac{1}{16} * \sqrt{28} * 500 * 642.5 = 106.24$ KN

$V_{s\,min} = \frac{1}{3}\ b_w\, d = \frac{1}{3} * 500 * 642.5 = 107.08$ KN Controls

Use two legs with $Av = 2 \times (\pi \times 10^2 / 4) = 157\ mm^2$

$$S = \frac{Av * f_y * d}{V_{s\,min}} = \frac{157 * 414 * 642.5}{107.08 * 10^3} = 390\ mm$$

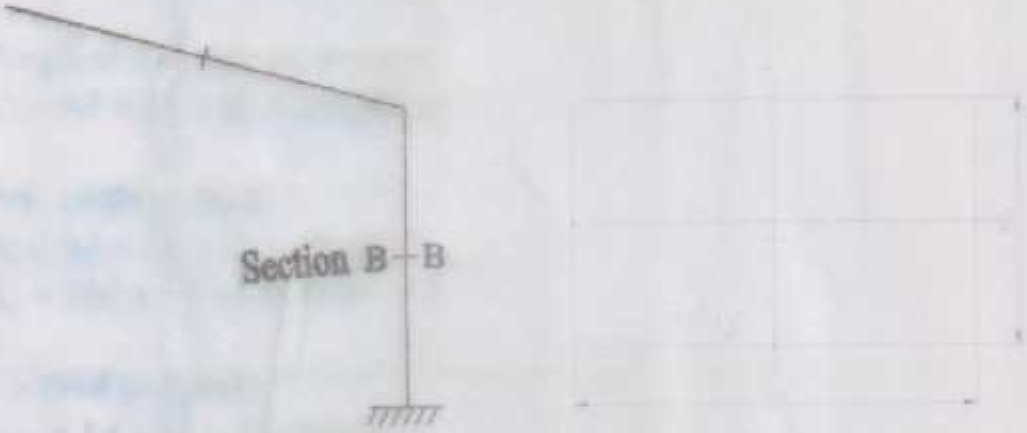
$$S = \frac{642.5}{2} = 321.25\ mm$$

$S \leq d/2$

$S \leq 600$ mm

Select Ø10/30 cm

4.1.3.5 Design of Exterior frame column



Column section (B-B) for Exterior Frame

Stability index:

Δ_o= Laterl deflection for ultimate case

$$Q = \frac{Pu \times \Delta_o}{V_u \times L_c} = \frac{215.25 \times 0.005}{2.4 \times 3.5} = 0.194 > 0.05$$

→Unbraced column

Slenderness:

$$\frac{KL}{r} \leq 22$$

System about x-axis

$$\frac{KL}{r} \leq 22 = \frac{2 \times 280}{0.3 \times 50} = 37.3 > 22 \rightarrow \text{Long Column}$$

System about y-axis

$$\frac{KL}{r} \leq 22 = \frac{2 \times 280}{0.3 \times 80} = 23.3 > 22 \rightarrow \text{Long Column}$$

Bresler equation:

$$\frac{1}{P_n} = \frac{1}{P_{nx}}$$

$$P_n = P_{nx}$$

By using the interaction diagram:

System about y-axis (in x –direction)

$$e = \frac{mu}{P_u} = \frac{403.88}{215.25} = 1.876\ \mathrm{m}$$

$$\frac{e}{h} = \frac{1.876}{0.8} = 2.34$$

$$\frac{\gamma}{h} = \frac{800 - 50 - 20 - 25}{800} = 0.88$$

$$\frac{\emptyset \times p_n}{A_g} = \frac{p_u}{A_g} = \frac{215.25/1000}{(0.5 \times 0.8)} = 0.538\ \text{mpa} = 0.08\ \text{ksi}$$

$$\frac{\emptyset \times m_n}{A_g \times h} = \frac{m_u}{A_g \times h} = \frac{403.88/1000}{(0.5 \times 0.8 \times 0.8)} = 1.26\ \text{mpa} = 0.18\ \text{ksi}$$

Simulate these values on the interaction diagram:

For $\frac{\gamma}{h} = 0.75 \rightarrow \rho_g = \rho_{g\,min} = 0.01$

For $\frac{\gamma}{h} = 0.90 \rightarrow \rho_g = \rho_{g\,min} = 0.01$

By interpolation ρ_g at $\frac{\gamma}{h} = 0.88 \rightarrow \rho_g = 0.01$

$As_{req} = \rho_g \times A_g = 0.01 \times 500 \times 800 = 4000\ \text{mm}^2$

$$n = \frac{4000}{254} = 15.7$$

Select 16Ø18

We try to design the column as flexural member and the result was 6Ø18 ,which is less than 16Ø18.

Design of stirrups:

Construction rules from ACI for tied reinforced section:

For diameter of ties:

For $\emptyset_{\text{Vertical bars}} = 18$ mm → Select $\emptyset_{\text{Stirrups}} = 8$ mm

Spacing of ties:

$S \leq 16 \times \emptyset_{\text{vertical bars}} = 16 \times 18 = 288$ mm

$S \leq 48 \times \emptyset_{\text{stirrups}} = 48 \times 8 = 384$ mm

$S \leq$ minimum dimenstion if colummn $= 500$ mm

$S = 250$ mm → **controls**

4.1.4 Design of Tie Beam

The tie beam will tie the footing with the adjacent footing, and in the same time carries the plain concrete wall above it.

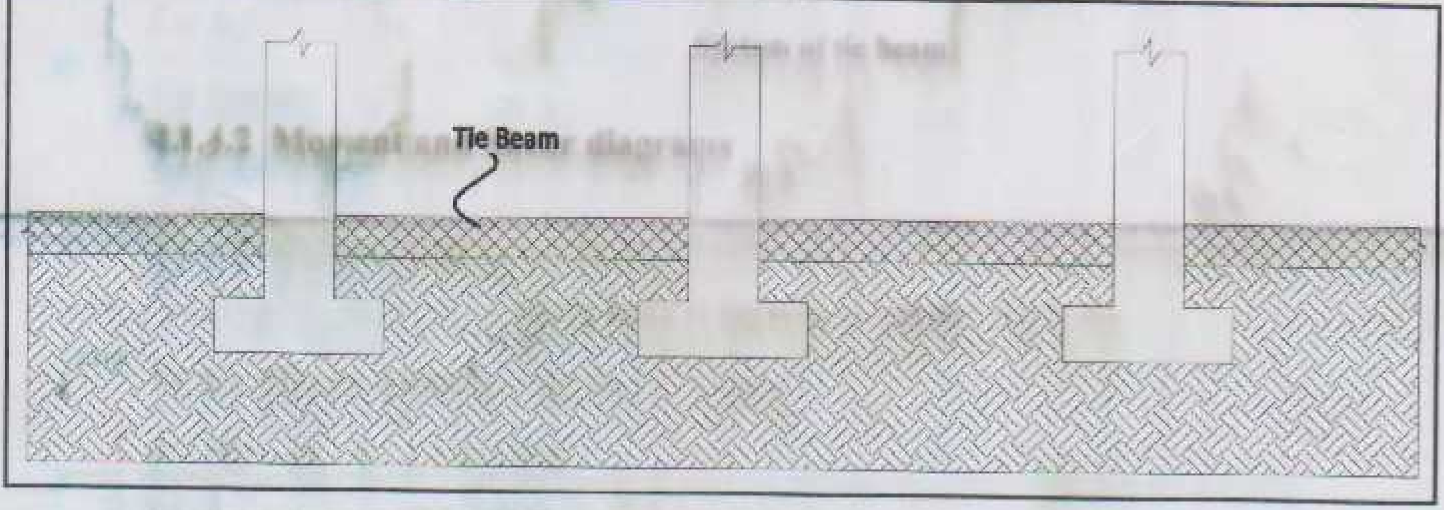


Figure (4-24): **Tie longitudinal section tire beam and footings**

4.1.4.1 Calculation of loads

Weight of wall over the tie beam

$q_d = 0.2 \times 2.5 \times 25 = 12.5\ KN/m$

Self-weight of beam:

$q_d = 0.2 \times 0.3 \times 25 = 1.5\ KN/m$

$q_d = 12.5 + 1.5 = 14\ KN/m$

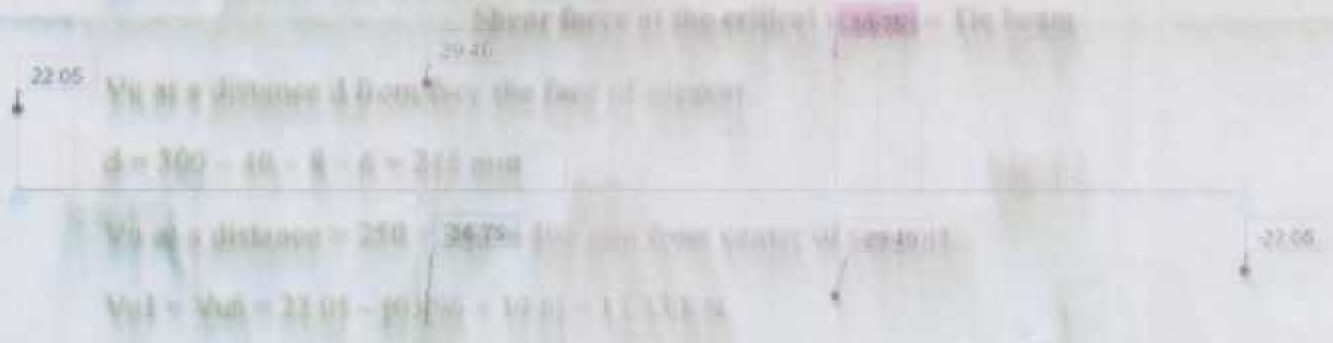
Factored dead load :

$q_{du} = 1.4 \times 14 = 19.6\ KN/m$

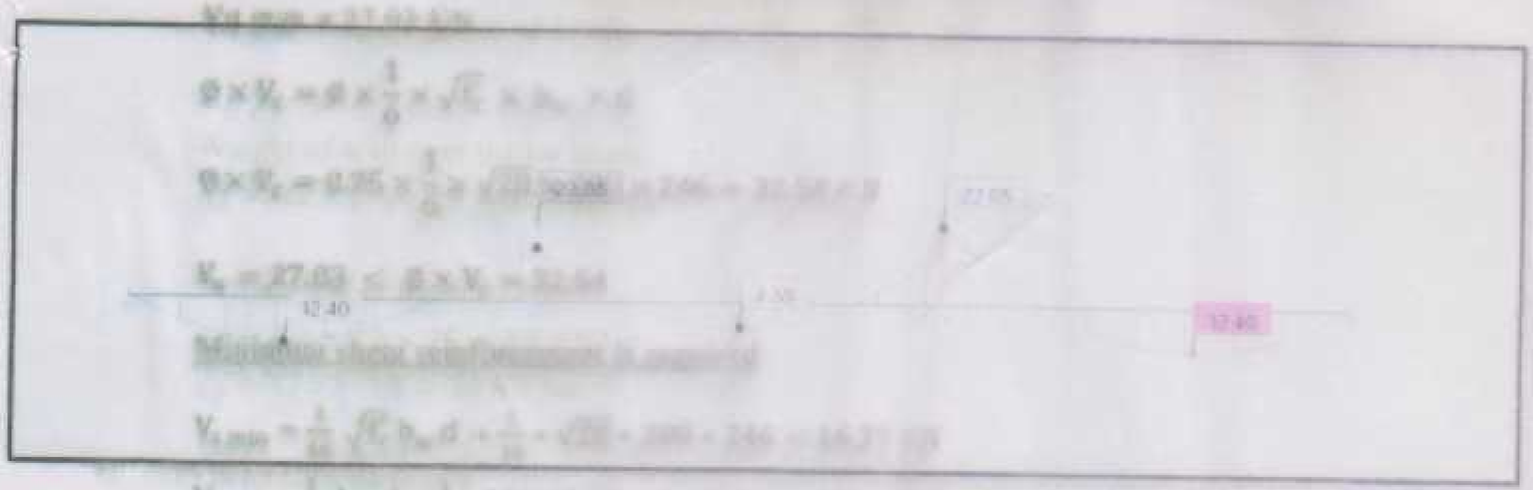
$q_{du} = 19.6$ KN\m

System of tie beam

4.1.4.2 Moment and Shear diagrams

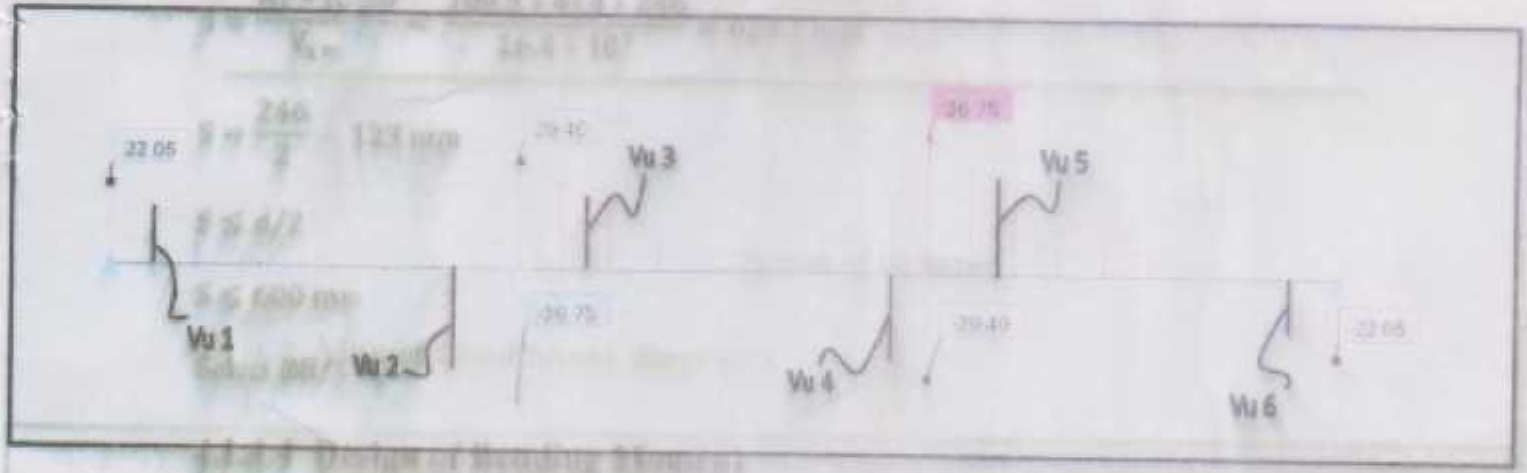


Tie beam - Shear Envelop Diagram



Tie beam - Moment Envelop Diagram

4.1.4.3 Design of Shear



Shear force at the critical section – Tie beam

Vu at a distance d from face the face of support.

$d = 300 - 40 - 8 - 6 = 246$ mm

Vu at a distance $= 250 + 246 = 496$ mm from center of support.

$Vu1 = Vu6 = 22.05 - [0.496 \times 19.6] = 12.33$ KN

$Vu2 = Vu5 = 36.75 - [0.496 \times 19.6] = 27.03$ KN

$Vu3 = Vu4 = 29.4 - [0.496 \times 19.6] = 19.68$ KN

Vu max = 27.03 KN

$$\emptyset \times V_c = \emptyset \times \frac{1}{6} \times \sqrt{f_c} \times b_w \times d$$

$$\emptyset \times V_c = 0.75 \times \frac{1}{6} \times \sqrt{28} \times 200 \times 246 = 32.54\ KN$$

$$V_u = 27.03 < \emptyset \times V_c = 32.54$$

Minimum shear reinforcement is required

$$V_{s\,min} = \frac{1}{16}\sqrt{f_c}\, b_w\, d = \frac{1}{16} * \sqrt{28} * 200 * 246 = 16.27\ KN$$

$$V_{s\,min} = \frac{1}{3}\ b_w\, d = \frac{1}{3} * 200 * 246 = 16.4\ KN$$ Controls

Use two legs with $Av = 2 * (\pi * 8^2 / 4) = 100.5\ mm^2$

$$S = \frac{Av * f_y * d}{V_{s\,min}} = \frac{100.5 * 414 * 246}{16.4 * 10^3} = 624.1\ mm$$

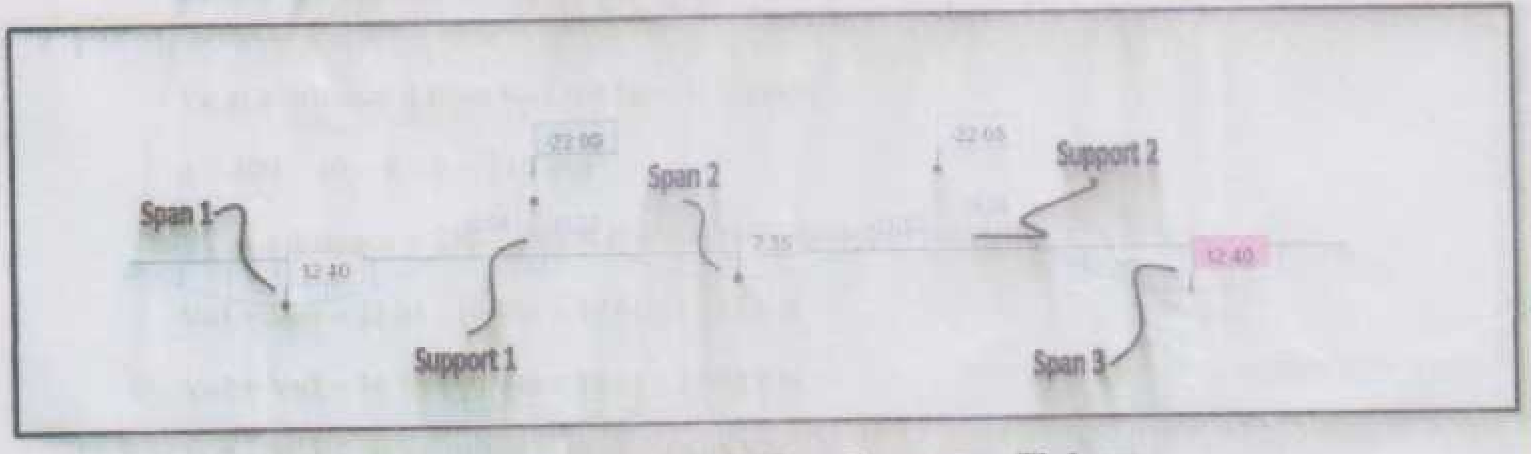
$$S = \frac{246}{2} = 123\ mm$$

$S \leq d/2$

$S \leq 600$ mm

Select Ø8/10 cm

4.1.4.4 Design of Bending Moment



Moment at the face of supports – Tie beam

Design for positive moment:

Design of span (1) and (3)

$M_u = 12.4$ KN.m

$$Kn = \frac{M_u/\emptyset}{bd^2} = \frac{12.4 \times 10^6/0.9}{200 \times (246)^2} = 1.14 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{414}{0.85 * 28} = 17.4$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{17.4}\left(1 - \sqrt{1 - \frac{2 \times 17.4 \times 1.14}{414}}\right) = 0.00282$$

$$As_{req} = \rho \times b \times d = 0.00282 * 200 * 246 = 138.89 \text{ mm}^2$$

$$As_{min} = 0.25 * \frac{\sqrt{f_c}}{f_y} * b * d = 0.25 \times \frac{\sqrt{28}}{414} \times 200 \times 246 = 157.21 \text{ mm}^2$$

Not less than:

$$As_{min} = \frac{1.4}{f_y} \times b \times d = \frac{1.4}{414} \times 200 \times 246 = 166.38 \text{ mm}^2$$

$$As_{req} = 138.89 \text{ mm}^2 < As_{min} = 166.38 \text{ mm}^2$$

$$As_{req} = 166.38 \text{ mm}^2$$

Select Ø12 mm . With $As = \frac{12^2 \times \pi}{4} = 113.04 \text{ mm}^2$

$$n = \frac{166.38}{113.04} = 1.47 \approx 2\emptyset 12 \text{ bars}$$

Check: strain

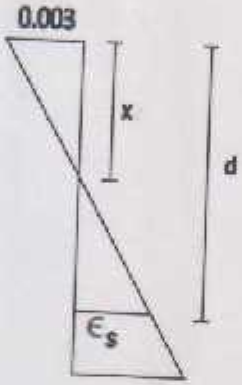
T = C

As $f_y = 0.85\, f_c'$ a b

$2 \times 113.04 \times 414 = 0.85 \times 28 \times a \times 200$

$a = 19.66$ mm

$x = \frac{19.66}{0.85} = 23.13$ mm



$\frac{0.003}{x} = \frac{0.003+\epsilon_s}{d}$

$\frac{0.003}{23.13} = \frac{0.003+\epsilon_s}{246}$

$\epsilon_x = 0.0289 > 0.005$

So → $\emptyset = 0.9 \rightarrow O.K$

Design of span (2)

$M_u = 7.35$ KN.m

$Kn = \frac{M_u/\emptyset}{bd^2} = \frac{7.35 \times 10^6/0.9}{200 \times (246)^2} = 0.675$ Mpa

$m = \frac{f_y}{0.85 f_c'} = \frac{414}{0.85 * 28} = 17.4$

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{17.4}\left(1 - \sqrt{1 - \frac{2 \times 17.4 \times 0.675}{414}}\right) = 0.00165$

$As_{req} = \rho \times b \times d = 0.00165 \times 200 \times 246 = 81.36\ mm^2$

$As_{min} = 0.25 * \frac{\sqrt{f_c'}}{f_y} * b * d = 0.25 \times \frac{\sqrt{28}}{414} \times 200 \times 246 = 157.21\ mm^2$

Not less than:

$As_{min} = \frac{1.4}{f_y} * b * d = \frac{1.4}{414} \times 200 \times 246 = 166.38\ mm^2$

$As_{req} = 81.36\ mm^2 < As_{min} = 166.38\ mm^2$

$As_{req} = 166.38\ mm^2$

Select $\emptyset 12$ mm with $As = \frac{12^2 \times \pi}{4} = 113.04\ mm^2$

$n = \frac{166.38}{113.04} = 1.47 \approx 2\emptyset 12$ bars

Check: strain

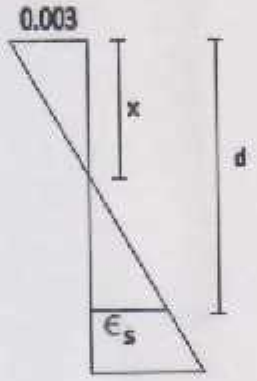
$T = C$

$As\, f_y = 0.85\, f_c'\, a\, b$

$2 \times 113.04 \times 414 = 0.85 \times 28 \times a \times 200$

$a = 19.66$ mm

$x = \frac{19.66}{0.85} = 23.13$ mm



$\frac{0.003}{x} = \frac{0.003+\epsilon_s}{d}$

$\frac{0.003}{23.13} = \frac{0.003+\epsilon_s}{246}$

$\epsilon_s = 0.0289 > 0.005$

So → $\emptyset = 0.9 \rightarrow O.K$

Design for negative moment:

Design of support (1) and (2)

$M_u = -19.04$ KN.m

$Kn = \frac{M_u/\emptyset}{bd^2} = \frac{19.04 \times 10^6/0.9}{200 \times (246)^2} = 1.75$ Mpa

$m = \frac{f_y}{0.85 f_c'} = \frac{414}{0.85 * 28} = 17.4$

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{17.4}\left(1 - \sqrt{1 - \frac{2 \times 17.4 \times 1.75}{414}}\right) = 0.0044$

$As_{req} = \rho \times b \times d = 0.0044 \times 200 \times 246 = 216.48\ mm^2$

$$As_{min} = 0.25 * \frac{\sqrt{f_c}}{f_y} * b * d = 0.25 \times \frac{\sqrt{28}}{414} \times 200 \times 246 = 157.21 \text{ mm}^2$$

Not less than:

$$As_{min} = \frac{1.4}{f_y} * b * d = \frac{1.4}{414} \times 200 \times 246 = 166.38 \text{ mm}^2$$

$$As_{req} = 216.48 \text{ mm}^2 > As_{min} = 166.38 \text{ mm}^2$$

<u>**Select**</u> <u>Ø12 mm.</u> with $As = \frac{12^2 \times \pi}{4} = 113.04 \text{ mm}^2$

$$n = \frac{216.48}{113.04} = 1.92 \approx 2Ø12 \text{ bars}$$

<u>**Check: strain**</u>

$$T = C$$

$$As\, f_y = 0.85\, f_c'\, a\, b$$

$$2 \times 113.04 \times 414 = 0.85 \times 28 \times a \times 200$$

$$a = 19.66 \text{ mm}$$

$$x = \frac{19.66}{0.85} = 23.13 \text{ mm}$$

$$\frac{0.003}{23.13} = \frac{0.003 + \epsilon_s}{246}$$

$$\frac{0.003}{23.13} = \frac{0.003 + \epsilon_s}{246}$$

$$\epsilon_s = 0.0289 > 0.005$$

$$\text{So} \rightarrow Ø = 0.9 \rightarrow O.K$$

<u>**As compression member**</u>

According to IBC 2012 the tied beam will be designed as a compression member or tension member by applying the maximum reaction on footing multiplies by 10 % , as axial force:

Pu = 428.9 * 0.1 = 42.89 KN

$$P_u = Ø \times P_n = 0.65 \times 0.80 \times A_g\left(0.85 \times f_{c'}(1 - \rho_g) + f_y \times \rho_g\right)$$
$$42.89 = 0.65 \times 0.80 \times 50000(0.85 \times 28(1 - \rho g) + 414 \times \rho g)$$

$\rho_g = -0.05818 < 0.01$ is less than the minimum reinforcement ratio

$As_{req} = 7.5\ cm^2$

As Tension member

$$A_{s\ required} = \frac{N}{f_y} = \frac{42.89x1000}{414} = 1.036\ cm^2$$

Select Ø12 *mm* with $As = \frac{1.2^2 \times \pi}{4} = 1.13\ cm^2$

$$n = \frac{7.5}{1.13} = 6.64 \approx 8Ø12\ \text{bars}$$

4.1.5 Design of footing

Subsurface condition

According to the soil report of the site (enclosed in the appendix) , the site surface soil is Sand-Silt Mixture with Gravel and the bearing capacity at the design depth is $1.6\frac{Kg}{cm^2} \approx 156.95\ \frac{KN}{m^2}$ and the specific weight of soil is $17.5\ \frac{KN}{m^3}$

Footing design

The foundation required to support the frame is a cantilever footing with dimension $L \times B$ where:

L: is the length of the footing that resists the moment.

B: is the width of the footing.

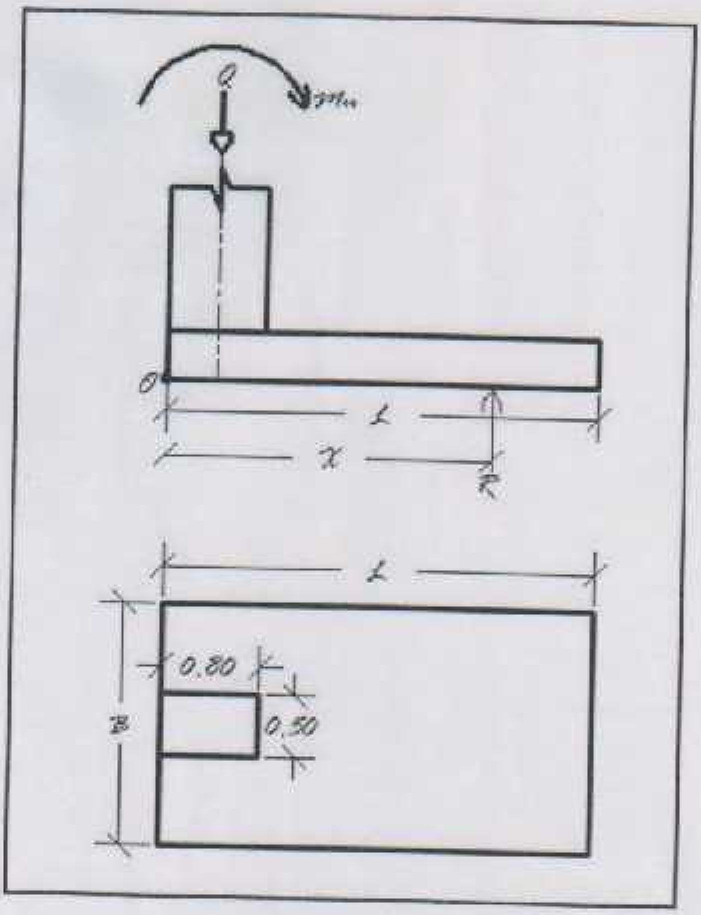


Figure (4-30): equilibrium system of footing

4.1.5.1 Design of Intermediate frame footing

The footing caries the frame column reaction in addition to the tie beam reaction (service load):

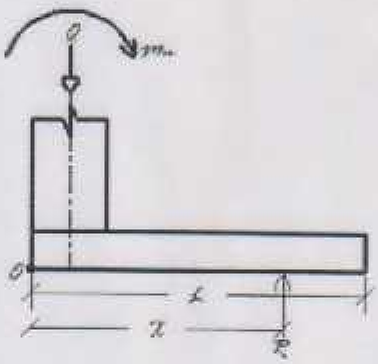
$Q = Q_{dead} + Q_{Live} + Q_{Snow} + Q_{Wind} + Q_{tie\ beam}$
$Q = 305.96\ KN$

Moment on the frame column:

$M_{survice} = 521.28\ KN.m$
These loads will appears as a reaction somewhere on the footing base (as shown)

Summation of vertical Force = zero

R = Q



By taking moment about point O:

$Q \times 0.4 + Ms = R \times X$
$305.96 \times 0.4 + 521.28 = 305.96 \times X \rightarrow X = 2.104\ m$
And the eccentricity:

$$e = \frac{L}{2} - X$$

Let's assume a value for **L** =2.5 m then:
$e = \frac{2.5}{2} - 2.104 = -0.764\ m$

The negative sign of e indicate that it exceeds the center of gravity of the footing base to the other side as shown in Figure (4-31).

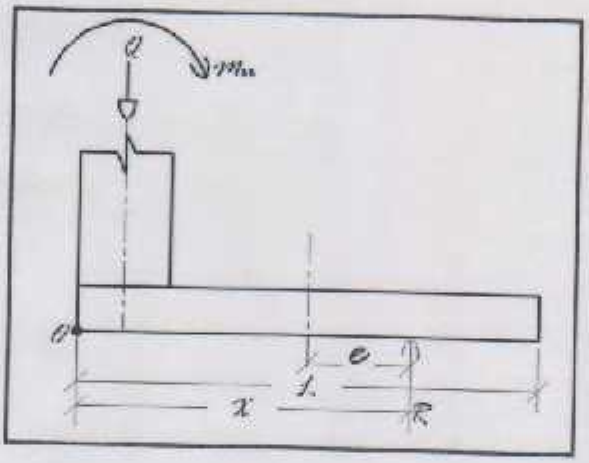


Figure (4-31): Eccentricity from center of footing - Intermediate frame footing

$$q_{\text{net-allowable}} = q_{\text{allowable}} - (q_{\text{self-weight}} + q_{\text{soil pressure}})$$

$$q_{\text{net-allowable}} = 156.95 - (25 \times 0.6_{(\text{assumed})}) + ((1 - 0.6) \times 17.5)$$

$$q_{\text{net-allowable}} = 134.95\ \frac{KN}{m^2}$$

Computing the width B

$$q_{\text{allowable}} = \frac{Q}{L \times B} \to B = \frac{Q}{L \times q_{\text{allowable}}} = \frac{R}{L \times q_{\text{allowable}}} = \frac{305.96}{136.45 \times 2.5}$$

$$B = \frac{305.96}{136.45 \times 2.5} = 0.897 \approx 1.2\text{ m}$$

Contact pressure

Since $e = 0.764 > \frac{L}{6} = \frac{2.5}{6} = 0.417m$ then q_{minimum} will be negative so the following equation for computing $q_{\max}$ is valid:

$$q_{\max} = \frac{4 \times Q}{3 \times B(L - 2 \times e)}$$

$$q_{\max} = \frac{4 \times 305.96}{3 \times 1.2(2.5 - 2 \times 0.764)} = 349.75\ KN$$

$$q_{\max} = 349.75 > q_{\text{net-allowable}} = 134.95\ \frac{KN}{m^2} \to Fail$$

Since this trail fails let's try some values for L:

Table (4-1): Trials for designing Intermediate frame footing dimensions

L(m)	B(m)	e(m)	$\frac{L}{6}$ (m)	q_{max} ($\frac{KN}{m^2}$)	$q_{minimum}$ ($\frac{KN}{m^2}$)	state
3	1	0.604	0.5	227.65	negative	227.65 > 134.95 → Fail
3.5	1.3	0.354	0.583	108.05	26.44	108.05 < 134.95 → pass

When $e = 0.354 < \frac{L}{6} = \frac{3.5}{6} = 0.58$ m then q_{max} and $q_{minimum}$ are given by the

Following equation:

$$q_{max} = \frac{Q}{B \times L}\left(1 + \frac{6e}{L}\right) = \frac{305.96}{1.3 \times 3.5}\left(1 + \frac{6 \times 0.354}{3.5}\right) = 108.05\ \frac{KN}{m^2}$$

$$q_{minimum} = \frac{Q}{B \times L}\left(1 - \frac{6e}{L}\right) = \frac{305.96}{1.2 \times 3.5}\left(1 - \frac{6 \times 0.354}{3.5}\right) = 28.64\ \frac{KN}{m^2}$$

thus the required dimension of the footing is $\mathbf{3.5 \times 1.3 (m)}$

Design of footing thickness and structural reinforcement:

Factored load:

$Q = 428.9KN$

$m_u = 748.6$ KN. m

Then the pressure under footing is

$q_{max} = 151.5\ \frac{KN}{m^2}$

$q_{minimum} = 37\ \frac{KN}{m^2}$

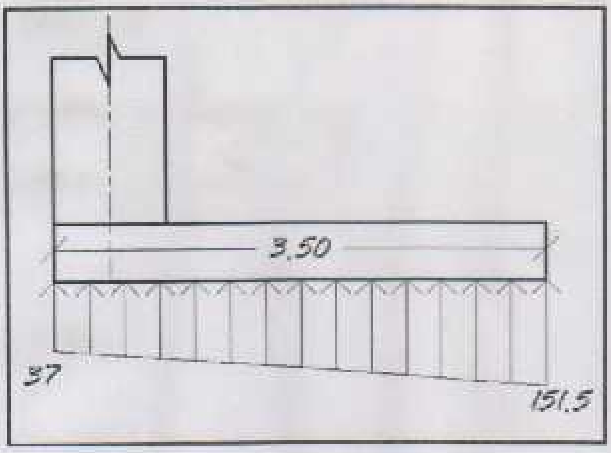


Figure (4-32): Contact Pressure diagram due to loading (factored) - Intermediate frame footing

Design of footing thickness: the thickness of footing will be selected so that punching shear failure does not occur. Assume $\emptyset_{max}$ main reinforcement is 25 mm and cover of 50 mm.

$d = 600 - 50 - 25 = 525$ mm

The footing will be design against one way shear and against punching shear.

Punching shear (Tow way shear):

Critical perimeter (b_o) will be at a distance $d/2$ from the face of the column

$b_o = 1.0625 \times 2 + 1.025 = 3.150$ m

Contact pressure at $d/2$ from the face of column (1.0625 m) is 71.76 $\frac{KN}{m^2}$, and the net factored shear to be transferred across the critical perimeter is:

$v_{u(net)} = 428.9 - 71.76 \times (1.0625 \times 1.025) = 350.751$ KN

The shear strength of concrete according to ACI is given by equation (11-31 to 11-33) as follows:

$V_c = 0.17\left(1 + \frac{2}{\beta}\right)\sqrt{f_c'}\, b_o\, d$ (11-31)

$V_c = 0.083\left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f_c'}\, b_o\, d$ (11-32)

$V_c = 0.33\sqrt{f_c'}\, b_o\, d$ (11-32)

The minimum value of the previous equations controls.

$\beta = \frac{80}{50} = 1.6$ $\alpha_s = 30$ (edge column)

The third equation controls:

$V_c = 0.33 \times \sqrt{28} \times 3510 \times 525 = 3217.8$ KN

$\emptyset \times V_c = 0.75 \times 3217.8 = 2413.35$ KN > 359.81 O.K

The footing thickness is adequate against two way shear.

One way shear: The critical section for checking one-way shear strength that is at a distance d from the face is shown in Fig. ( ), the pressure at d from the face (1.325 m) is 80.35 $\frac{KN}{m^2}$ and at the very far end of the footing is 151.5 $\frac{KN}{m^2}$ so that → V_u :

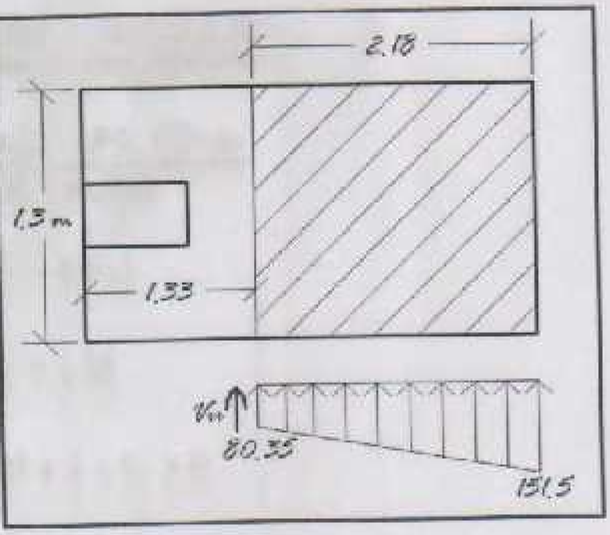


Figure (4-33): critical section for checking one-way - Intermediate frame footing

$V_u = \left(\frac{80.35+151.5}{2}\right) \times 2.175 \times 1.3 = 327.8$ KN

$\emptyset \times V_c = \emptyset \times \frac{1}{6} \times \sqrt{f_c'} \times b_w \times d = 0.75 \times \frac{1}{6} \times \sqrt{28} \times 1300 \times 525 = 451.43$ KN

$\emptyset \times V_c = 451.43 > V_u = 327.8$ → No shear reinforcement

Thickness of footing is adequate against one way shear

Design of footing against bending moment:

The critical section for bending moment that is at the face as shown in Fig. ( ), the pressure at the face (0.8 m) is 63.2 $\frac{KN}{m^2}$ and at the very end of the footing is 151.5 $\frac{KN}{m^2}$ so that Mu :

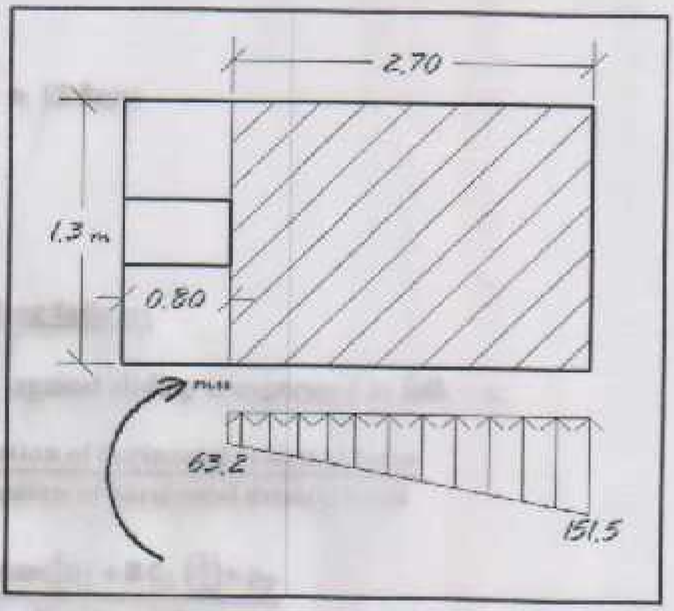


Figure (4-34): critical section for design Moment - Intermediate frame footing

$$M_u = \left(63.2 \times 1.3 \times 2.7 \times \tfrac{2.7}{2}\right) + \left(88.3 \times 1.3 \times 2.7 \times \tfrac{1}{2} \times 1.8\right) = 578.4 \text{ KN.m}$$

$$Kn = \frac{M_u/\emptyset}{bd^2} = \frac{578.4 \times 10^6/0.9}{1300 \times (525)^2} = 1.79 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{414}{0.85 \times 28} = 17.4$$

$$\rho_{required} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{17.4}\left(1 - \sqrt{1 - \frac{2 \times 17.4 \times 1.79}{414}}\right) = 0.0045$$

$$A_{s\,required} = \rho_{required} \times b \times d = 0.0045 \times 1300 \times 525 = 3071.1 \text{ mm}^2$$

Select Ø25 mm, With $As = \frac{25^2 \times \pi}{4} = 490.6 \text{ mm}^2$

$n = \frac{3071.1}{490.6} = 6.26 \approx 7Ø25$ bars

To cover the secondary moment, shrinkage and temperature is required:

$As_{min} = 0.0018 \times 1300 \times 525 = 1228.5$ mm2

Select Ø12 mm

$n = \frac{1228.5}{113} = 10.8 \approx 11\ bars$

Use Ø12 /25 cm

Footing stability:

Check against sliding failure:

The factor of safety against sliding is expressed as follows:

$$F.S_{sliding} = \frac{\text{sumation of horizontal resisting foces}}{\text{sumation of horizontal driving foces}}$$

$$F.S_{sliding} = \frac{\varepsilon V \times \tan(\frac{2}{3}\emptyset) + B\,C_2\left(\frac{2}{3}\right) + p_p}{R_x + p_a \cos\beta}$$

$$K_a = \frac{1-\sin\emptyset}{1+\sin\emptyset} = \frac{1-\sin 13}{1+\sin 13} = 0.633$$

$$K_p = \frac{1+\sin\emptyset}{1-\sin\emptyset} = \frac{1+\sin 13}{1-\sin 13} = 1.58$$

$$p_p = \frac{1}{2} \times \gamma_2 \times K_p \times D^2 + 2 \times C_2 \times D \times \sqrt{K_p}$$

$$p_p = \frac{1}{2} \times 17.5 \times 1.58 \times 0.4^2 + 2 \times 11.77 \times 0.4 \times \sqrt{1.58} = 14.05$$

$$p_a = \frac{1}{2} \times \gamma_1 \times K_a \times H = \frac{1}{2} \times 17.5 \times 0.633 \times 1 = 5.54$$

εV = weight of soil above footing + weight of footing + Q_u

$$\varepsilon V = 2.7 \times 17.5 \times 1.3 \times 0.4 + 3.5 \times 25 \times 1.3 \times 0.6 + 428.9 = 521.72\ KN$$

$$F.S_{sliding} = \frac{521.72 \times \tan(\frac{2}{3} \times 13) + 3.5 \times 11.77 \times (\frac{2}{3}) + 14.05}{4.8 + 5.54 \times \cos 0} = 11.7 > 1.5 \rightarrow O.K$$

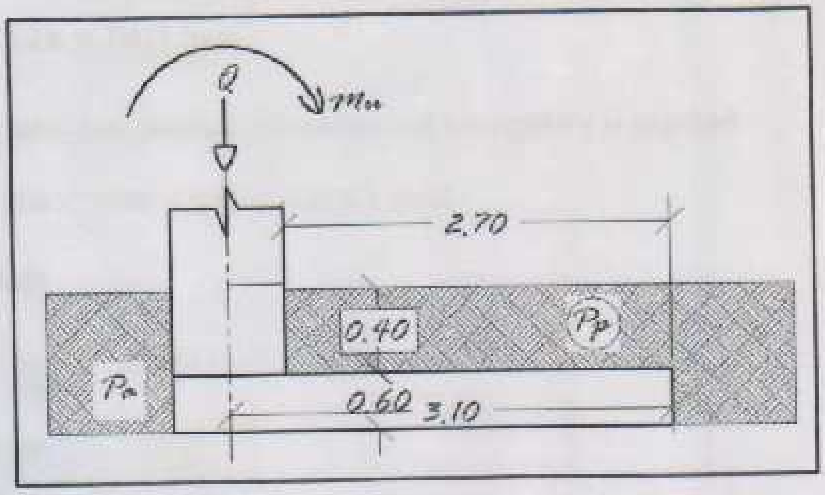


Figure (4-35): **Forces for check against sliding failure - Intermediate frame footing**

Check against overturning failure:

The moment will be taken about point O as shown

$$F.s_{overturning} = \frac{\text{Resisiting moment}}{\text{overturning moment}}$$

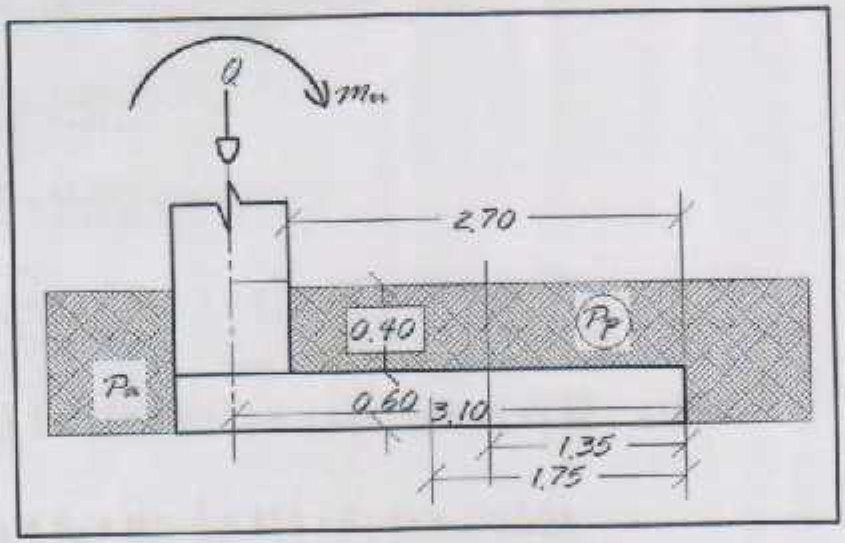


Figure (4-36): **Forces for Check against overturning failure - Intermediate frame footing**

$$F.s_{\,overturning} = \frac{68.25 \times 1.75 + 24.57 \times 1.35 + 428.9 \times 3.1}{748.6} = 1.98 > 1.5 \rightarrow O.K$$

4.1.5.2 Design of Exterior frame footing

The footing caries the frame column reaction in addition to the tie beam reaction (service load):

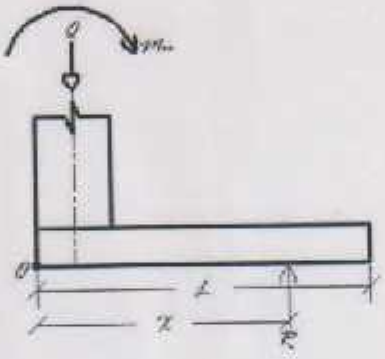
$Q = Q_{dead} + Q_{Live} + Q_{Snow} + Q_{Wind} + Q_{tie\ beam}$
$Q = 204.82\ KN$

Moment on the frame column:

$M_{survice} = 285.28\ KN.m$
These loads will appears as a reaction somewhere on the footing base (as shown)

Summation of vertical Force = zero

R = Q



By taking moment about point O:

$Q \times 0.4 + Ms = R \times X$
$204.82 \times 0.4 + 285.28 = 204.82 \times X \rightarrow X = 1.79\ m$
And the eccentricity:

$e = \frac{L}{2} - X$

Let's assume a value for L =2.5 m then:
$e = \frac{2.5}{2} - 1.79 = -0.543\ m$

The negative sign of e indicate that it exceeds the center of gravity of the footing base to the other side as shown in fig ()

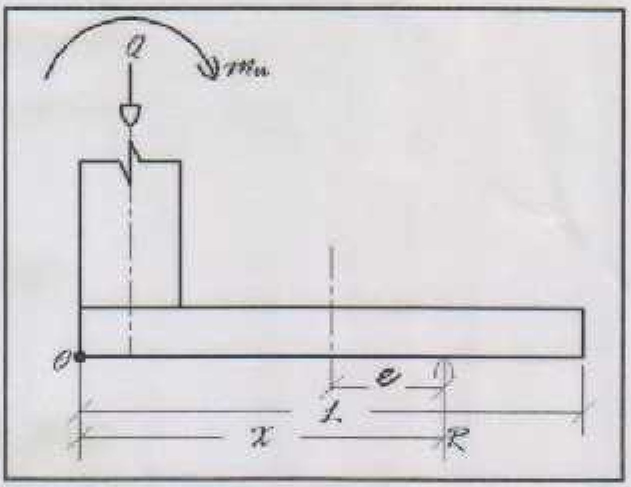


Figure (4-37): Eccentricity from center of footing - Exterior frame footing

$q_{\text{net-allowable}} = q_{\text{allowable}} - (q_{\text{self-weight}} + q_{\text{soil pressure}})$

$q_{\text{net-allowable}} = 156.95 - (25 \times 0.4_{(\text{assumed})}) + ((1 - 0.4) \times 17.5)$

$q_{\text{net-allowable}} = 136.45 \dfrac{KN}{m^2}$

Computing the width B

$$q_{\text{allowable}} = \frac{Q}{L \times B} \to B = \frac{Q}{L \times q_{\text{allowable}}} = \frac{R}{L \times q_{\text{allowable}}} = \frac{204.82}{136.45 \times 2.5}$$

$$B = \frac{204.82}{136.45 \times 2.5} = 0.6 \approx 1.0 \text{ m}$$

Contact pressure

Since $e = 0.543 > \frac{L}{6} = \frac{2.5}{6} = 0.417m$ then q_{minimum} will be negative so the following equation for computing q_{max} is valid:

$$q_{\text{max}} = \frac{4 \times Q}{3 \times B(L - 2 \times e)}$$

$$q_{\text{max}} = \frac{4 \times 204.85}{3 \times 1.0(2.5 - 2 \times 0.543)} = 193.16 \, KN$$

$q_{\text{max}} = 193.16 > q_{\text{net-allowable}} = 136.45 \frac{KN}{m^2} \to Fail$

Since this trail fails let's try some values for L :

L(m)	B(m)	e(m)	$\frac{L}{6}$ (m)	q_{max} $(\frac{KN}{m^2})$	$q_{minimum}$ $(\frac{KN}{m^2})$	state
3	1	0.29	0.5	107.87	28.67	107.87 < 136.45 → pass

When $e = 0.29 < \frac{L}{6} = \frac{3.5}{6} = 0.58$ m then q_{max} and $q_{minimum}$ are given by the

following equation:

$$q_{max} = \frac{Q}{B \times L}\left(1 + \frac{6e}{L}\right) = \frac{204.82}{1 \times 3}\left(1 + \frac{6 \times 0.29}{3}\right) = 107.87\ \frac{KN}{m^2}$$

$$q_{minimum} = \frac{Q}{B \times L}\left(1 - \frac{6e}{L}\right) = \frac{204.82}{1 \times 3}\left(1 - \frac{6 \times 0.29}{3}\right) = 28.64\ \frac{KN}{m^2}$$

thus the required dimension of the footing is $\mathbf{3.0 \times 1.0(m)}$

Design of footing thickness and structural reinforcement:

Factored load:

$Q = 281.4$ KN

$m_u = 403.88$ KN.m

Then the pressure under footing is

$q_{max} = 148.2\ \frac{KN}{m^2}$

$q_{minimum} = 39.4\ \frac{KN}{m^2}$

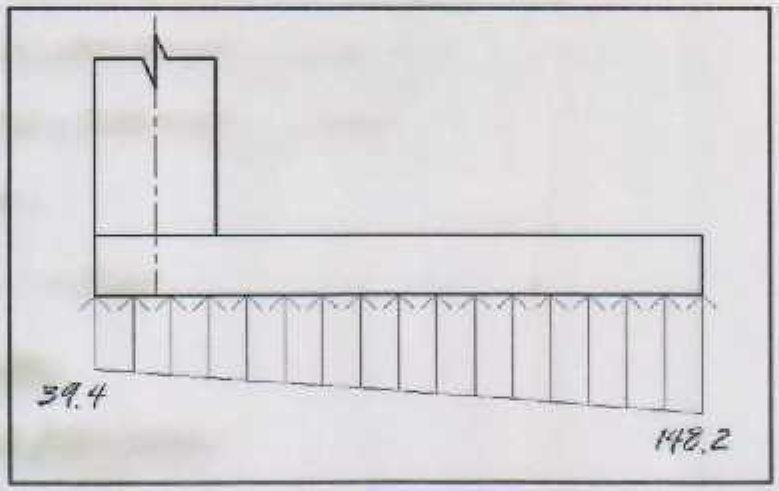


Figure (4-38): Contact Pressure diagram due to loading (factored) - Exterior frame footing

Design of footing thickness: the thickness of footing will be selected so that punching shear failure does not occur. Assume $\emptyset_{max}$ main reinforcement is 25 mm and cover of 40 mm.

$d = 400 - 40 - 25 = 335$ mm

The footing will be design against one way shear and against punching shear.

Punching shear (Tow way shear):

Critical perimeter (b_o) will be at a distance $d/2$ from the face of the column

$b_o = 0.968 \times 2 + 0.835 = 2.771$ m

Contact pressure at $d/2$ from the face of column (0.968 m) is 74.37 $\frac{KN}{m^2}$, and the net factored shear to be transferred across the critical perimeter is:

$v_{u(net)} = 281.4 - 74.37 \times (0.968 \times 0.835) = 221.29$ KN

The shear strength of concrete according to ACI is given by equation (11-31 to 11-33) as follows:

$V_c = 0.17\left(1 + \frac{2}{\beta}\right)\sqrt{f_c'}\, b_o\, d$ (11-31)

$V_c = 0.083\left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f_c'}\, b_o\, d$ (11-32)

$V_c = 0.33\sqrt{f_c'}\, b_o\, d$ (11-32)

The minimum value of the previous equations controls.

$\beta = \frac{80}{50} = 1.6 \qquad \alpha_s = 30$ (edge column)

The third equation controls:

$V_c = 0.33 \times \sqrt{28} \times 2771 \times 335 = 1620.97$ KN

$\emptyset \times V_c = 0.75 \times 1620.97 = 1215.72$ KN > 221.29 O.K

The footing thickness is adequate against two way shear.

One way shear: The critical section for checking one-way shear strength that is at a distance d from the face is shown in Fig. ( ), the pressure at d from the face (1.135 m) is 80.44 $\frac{KN}{m^2}$ and at the very far end of the footing is 148.2 $\frac{KN}{m^2}$ so that → V_u :

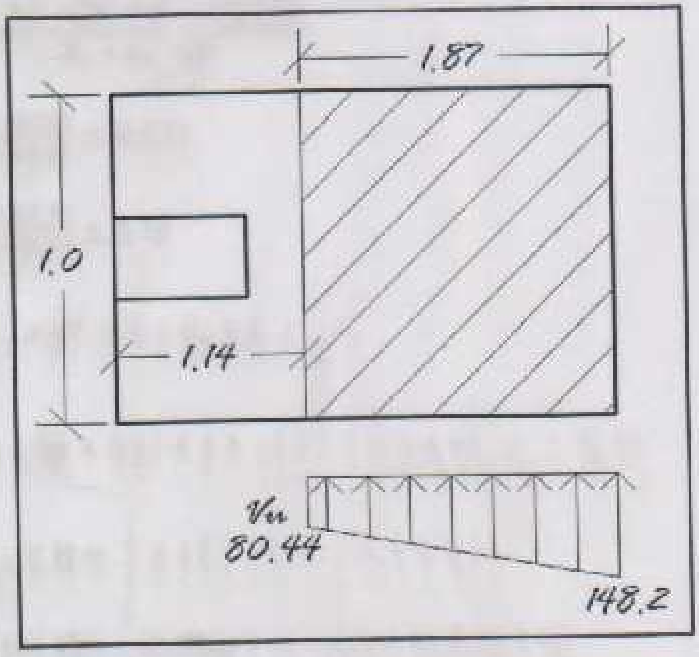


Figure (4-39): critical section for checking one-way - Exterior frame footing

$V_u = \left(\frac{80.44+148.2}{2}\right) \times 1.87 \times 1 = 213.78$ KN

$\emptyset \times V_c = \emptyset \times \frac{1}{6} \times \sqrt{f_{c'}} \times b_w \times d = 0.75 \times \frac{1}{6} \times \sqrt{28} \times 1000 \times 335 = 221.58$ KN

$V_u = 213.78$ KN $< 221.58 = 305.33$ KN → No Shear reinforcemet is required

Design of footing against bending moment:

The critical section for bending moment that is at the face as shown in Fig. ( ), the pressure at the face (0.8 m) is 68.27 $\frac{KN}{m^2}$ and at the very end of the footing is 148.2 $\frac{KN}{m^2}$ so that Mu :

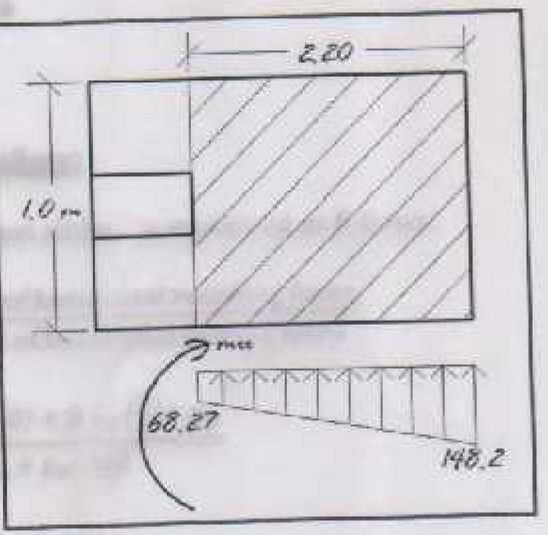


Figure (4-40): critical section for design Moment - Exterior frame footing

$$M_u = \left(68.27 \times 2.2 \times 1.0 \times \frac{2.2}{2}\right) + \left(109 \times 1.0 \times 2.2 \times \frac{1}{2} \times 1.47\right) = 341.5 \text{ KN.m}$$

$$Kn = \frac{M_u/\emptyset}{bd^2} = \frac{341.5 \times 10^6/0.9}{1000 \times (335)^2} = 3.38 \text{ Mpa}$$

$$m = \frac{f_y}{0.85f'_c} = \frac{414}{0.85 * 28} = 17.4$$

$$\rho_{required} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{17.4}\left(1 - \sqrt{1 - \frac{2 \times 17.4 \times 3.38}{414}}\right) = 0.0088$$

$$A_{s\ required} = \rho_{required} \times b \times d = 0.00884 \times 1000 \times 335 = 2963.03 \text{ mm}^2$$

Select Ø18 mm, With $As = \frac{18^2 \times \pi}{4} = 254.34 \text{ mm}^2$

$$n = \frac{2963.03}{254.34} = 11.65 \approx 12Ø18 \text{ bars}$$

To cover the secondary moment, shrinkage and temperature is required:

$As_{min} = 0.0018 \times 1000 \times 335 = 603$ mm2

Select Ø10 mm

$n = \frac{723.6}{78.5} = 7.7 \approx 8$ bars

Footing stability:

Check against sliding failure:

The factor of safety against sliding is expressed as follows:

$$F.S_{sliding} = \frac{\text{sumation of horizontal resisting foces}}{\text{sumation of horizontal driving foces}}$$

$$F.S_{sliding} = \frac{\varepsilon V \times \tan(\frac{2}{3}\emptyset) + B\,C_2\left(\frac{2}{3}\right) + p_p}{R_x + p_a \cos\beta}$$

$$K_a = \frac{1-\sin\emptyset}{1+\sin\emptyset} = \frac{1-\sin 13}{1+\sin 13} = 0.633$$

$$K_p = \frac{1+\sin\emptyset}{1-\sin\emptyset} = \frac{1+\sin 13}{1-\sin 13} = 1.58$$

$$p_p = \frac{1}{2} \times \gamma_2 \times K_p \times D^2 + 2 \times C_2 \times D \times \sqrt{K_p}$$

$$p_p = \frac{1}{2} \times 17.5 \times 1.58 \times 0.6^2 + 2 \times 11.77 \times 0.6 \times \sqrt{1.58} = 22.73$$

$$p_a = \frac{1}{2} \times \gamma_1 \times K_a \times H = \frac{1}{2} \times 17.5 \times 0.633 \times 1 = 5.54$$

εV = weight of soil above footing + weight of footing + Q_u

$$\varepsilon V = 2.2 \times 17.5 \times 1.0 \times 0.6 + 3.0 \times 25 \times 1.0 \times 0.4 + 281.4$$

$$\varepsilon V = 23.1 + 30 + 281.4 = 334.5 \text{ KN}$$

$$F.S_{sliding} = \frac{334.5 \times \tan(\frac{2}{3} \times 13) + 3.0 \times 11.77 \times (\frac{2}{3}) + 22.73}{2.4 + 5.54 \times \cos 0} = 12.2 > 1.5 \rightarrow \text{O.K}$$

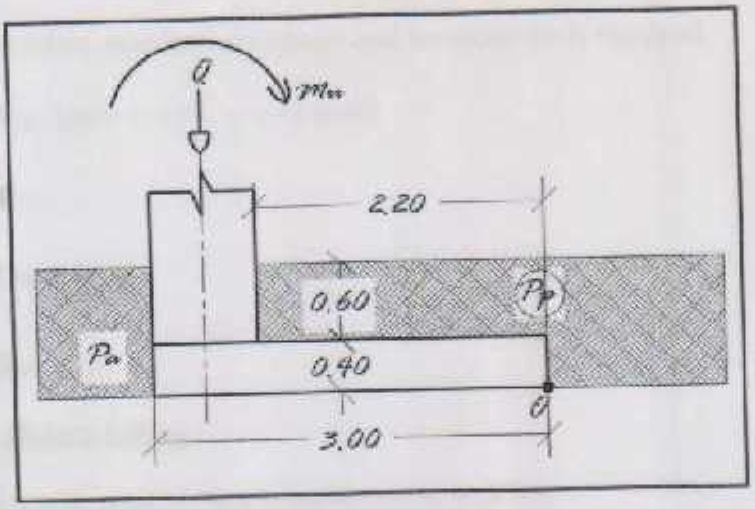


Figure (4-41): Forces for check against sliding failure - Exterior frame footing

Check against overturning failure:

The moment will be taken about point O as shown

$$F.S_{overturning} = \frac{\text{Resisiting moment}}{\text{overturning moment}}$$

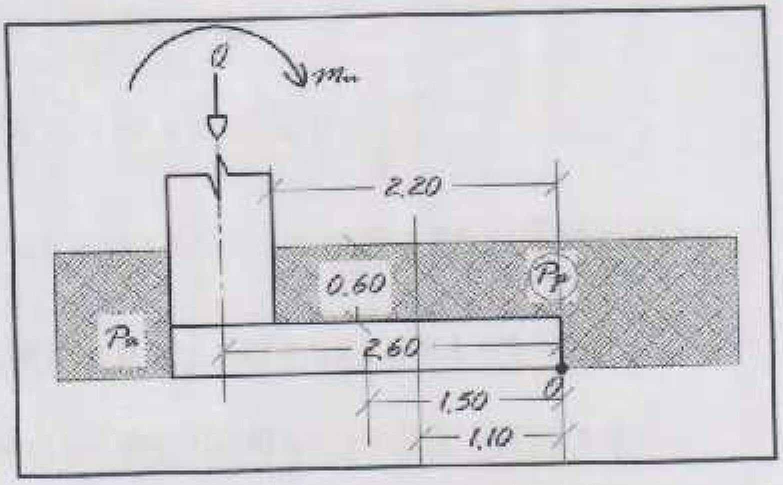


Figure (4-42): Forces for Check against overturning failure - Exterior frame footing

$$F.S_{overturning} = \frac{30 \times 1.5 + 23.1 \times 1.1 + 281.4 \times 2.6}{403.88} = 1.98 > 1.5 \rightarrow O.K$$

4.2 Design of Administration Building

For the administration building the concrete compression strength (f_c') is 24 Mpa and the tensile strength of reinforcement rebar (f_y) is 420 Mpa, and the density of concrete is 25 KN/m3.

4.2.1 Design of rib slab

4.2.1.1 Limitation of deflection

According to ACI the calculation of deflection can be neglected if the thickness of slab greater than h_{min}.

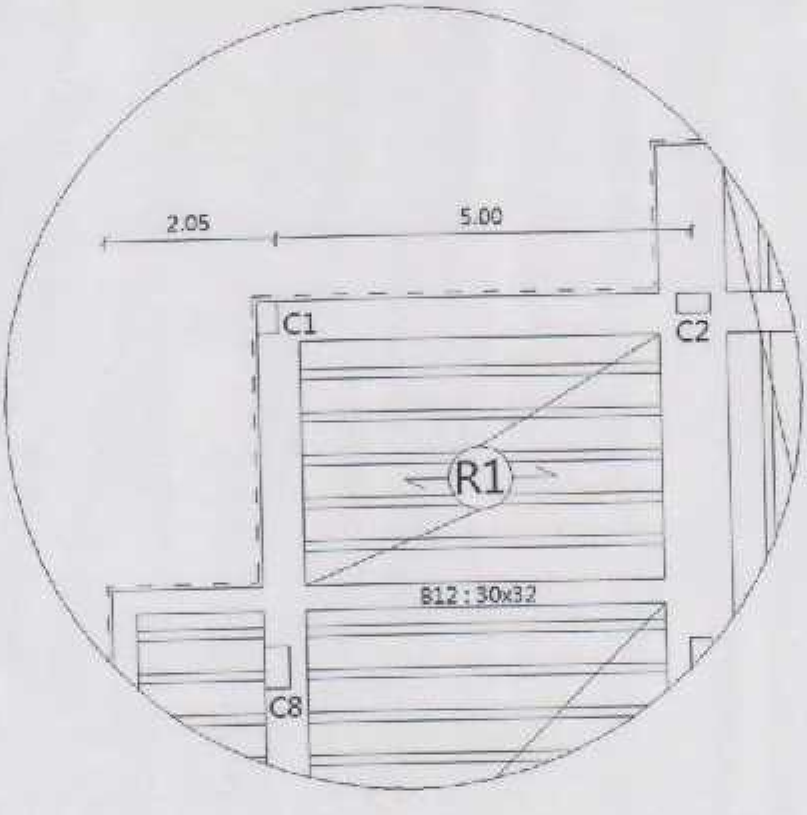


Figure (4-43): **Maximum rib spans (for one end continues)**

Max span for one end continues span = 5.0 m

Max span for simply supported = 5.0 m

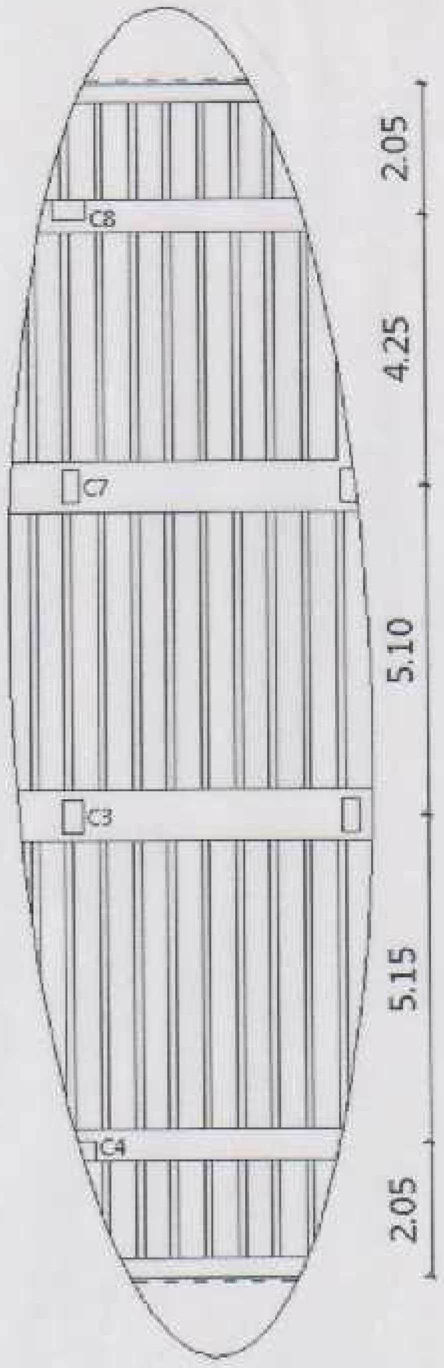


Figure (4-44): **Maximum rib spans (for both end continues)**

Max span for both end continues span = 5.15 m

For cantilevers = 2.05 m

According to ACI-Code-318-08 table 9.5(a), the minimum thickness:

- One end continuous: h_{min} = L / 18.5 = 500 / 18.5 = 27.03 cm
- Both end continuous h_{min} = L / 21 = 515 / 21 = 24.52 cm
- Simply supported h_{min} = L / 16 = 500 / 16 = 31.25 cm
- Cantilevers h_{min} = L / 8 = 205 / 8 = 25.63 cm

Select Block 24 cm with h_{slab} = 32 cm > h_{min} = 31.25 cm

h_{slab} = 32 cm ≤ 3.5 × width of rib = 3.5 × 12 = 42 cm

Thickness of topping = 8 cm

4.2.1.2 Design of topping

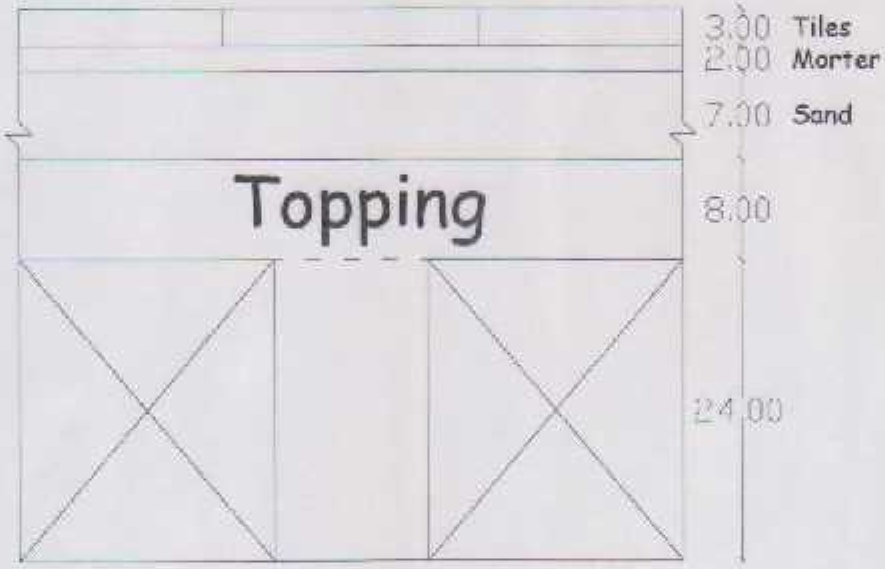


Figure (4-45): Typical section in topping

Dead loads

Tiles	23× 0.03= 0.69 KN/m²
Mortar	22 × 0.02 = 0.44 KN/m²
Sand	16× 0.07 = 1.12 KN/m²
Topping	25× 0.08 = 2.0 KN/m²
Partition	2.5 KN/m²

Total Dead Load = 0.69 + 0.44 + 1.12+ 2.5 + 2.5 = 6.75 KN/m²

For one meter strip:

Dead load = 6.75 × 1 = 6.75 KN/m

Live load = 2.5 × 1 = 2.5 KN/m

q_u = 1.2 × dead load + 1.6 × live load

= 12.1 KN/m

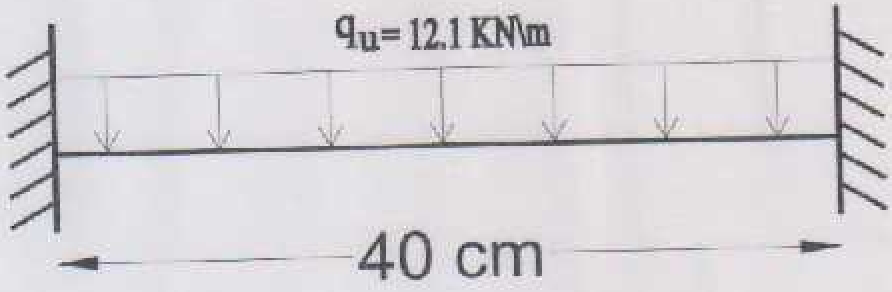


Figure (4-46): System of topping

Mu = (qu × L^2 / 12) = 12.1 × 0.4^2 / 12 = 0.161 KN.m

Vu = 12.1 × 0.2 = 2.42 KN

4.2.1.2.1 Design of Shear strength

Vu max = 2.42 KN.

$Vc = \frac{1}{6}\lambda\sqrt{f_c}bh = \frac{1}{6}\sqrt{24} \times 1000 \times 80 \times 10^{-3} = 65.32$ KN

Ø Vc = 0.75× 65.32 = 48.99 KN

Vu max = 2.42 KN << Ø Vc = 48.99

→No shear reinforcement is required.

4.2.1.2.2 Design of Bending Moment

Mu = 0.161 KN.m

Plain concrete section with b / h = 1000/80 cm

$$ØMn = 0.55 * 0.42 * \sqrt{f_c'} * \frac{bh^2}{6}$$

$$ØMn = 0.55 * 0.42 * \sqrt{24} * \frac{1000 * 80^2}{6} = 1.207 \text{ KN. M}$$

Ø Mn = 1.207 KN.m > Mu = 0.161 KN.m

No reinforcement is required

Minimum reinforcement for shrinkage and temperature

$As_{min} = 0.0018\ b\ h = 0.0018 * 1000 * 100 = 180\ mm^2$ *ACI-318-08 (7.12.2.1)*

Select mesh Ø8/20cm with $As = (\pi \times 8^2 / 4) \times 100/20 = 251.33mm^2$

4.2.1.3 Design of rib (5)

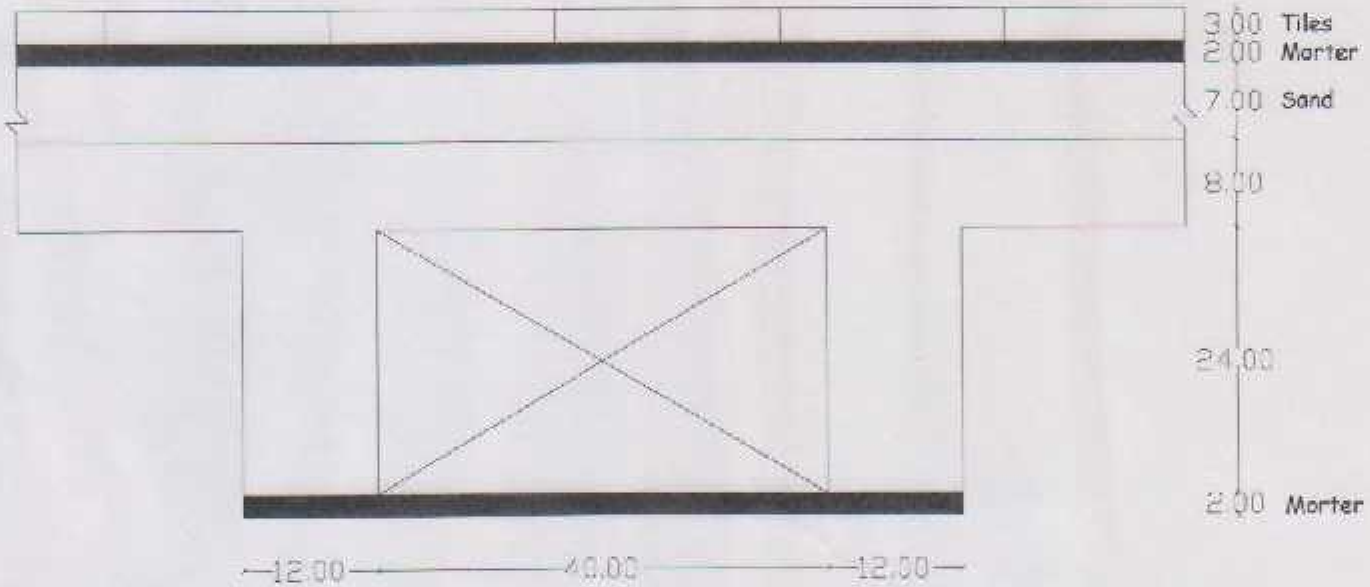


Figure (4-47): Typical section in Rib

Dead loads

Tiles	$23\times0.03\times0.52=0.36$ KN/m
Mortar	$22\times0.02\times0.52=0.23$ KN/m
Sand	$16\times0.07\times0.52=0.58$ KN/m
Topping	$25\times0.08\times0.52=1.04$ KN/m
Block	$15\times0.24\times0.4=1.44$ KN/m
Rib	$25\times0.12\times0.24=0.72$ KN/m
Plaster	$22\times0.02\times0.52=0.23$ KN/m
Partition	$2.5\times0.52=1.3$ KN/m

Total Dead Load = 0.36 + 0.23 + 0.58 + 1.04 + 1.44 + 0.72 + 0.23 + 1.3 = 5.9 KN/m

Live load = $2.5\times0.52=1.3$ KN/m

$q_u=1.2\times$ dead load + $1.6\times$ live load

= 9.16 KN/m

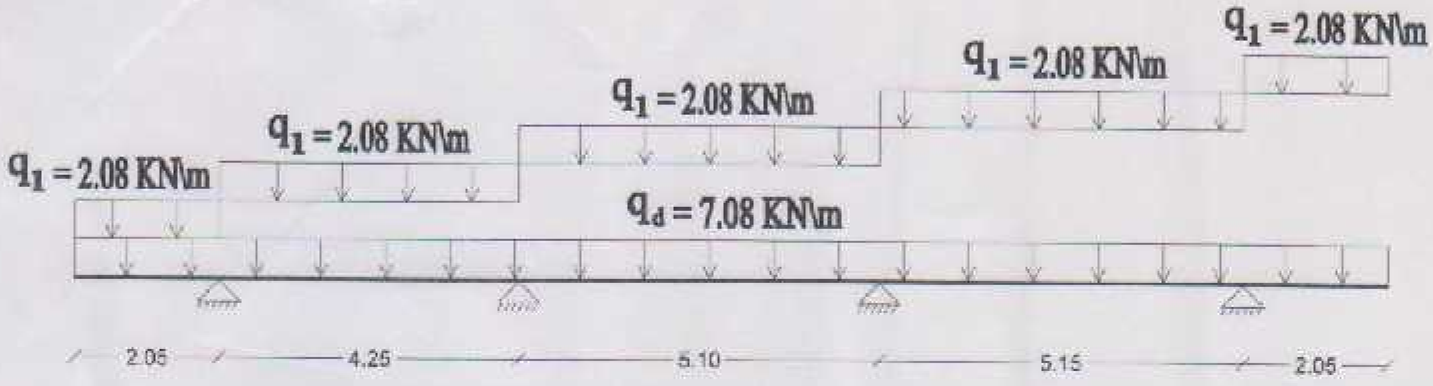


Figure (4-48): System of rib

4.2.1.3.1 Design of Shear

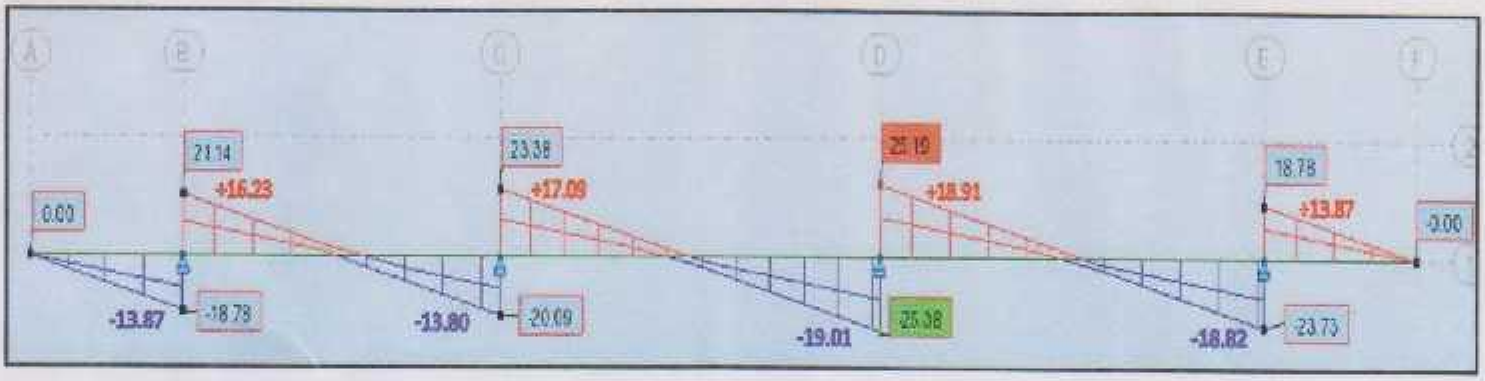


Figure (4-49): Shear force at the critical section – Rib

Max Vu at the critical section = 19.01 KN

$1.1\varnothing\, V_c = 1.1 * 0.75\, \frac{1}{6} * \sqrt{f_c'}\, b_w\, d$

d = 320 - 20 - 8 - 6 = 286 mm

$1.1\varnothing\, V_c = 1.1 * 0.75 * \frac{1}{6} * \sqrt{24}\ 120\ 286 = 23.12$ KN

Vu = 19.01 < 23.12 KN

→No shear reinforcement is required.

Select Ø8/25cm Montage

4.2.1.3.1 Design of bending moment

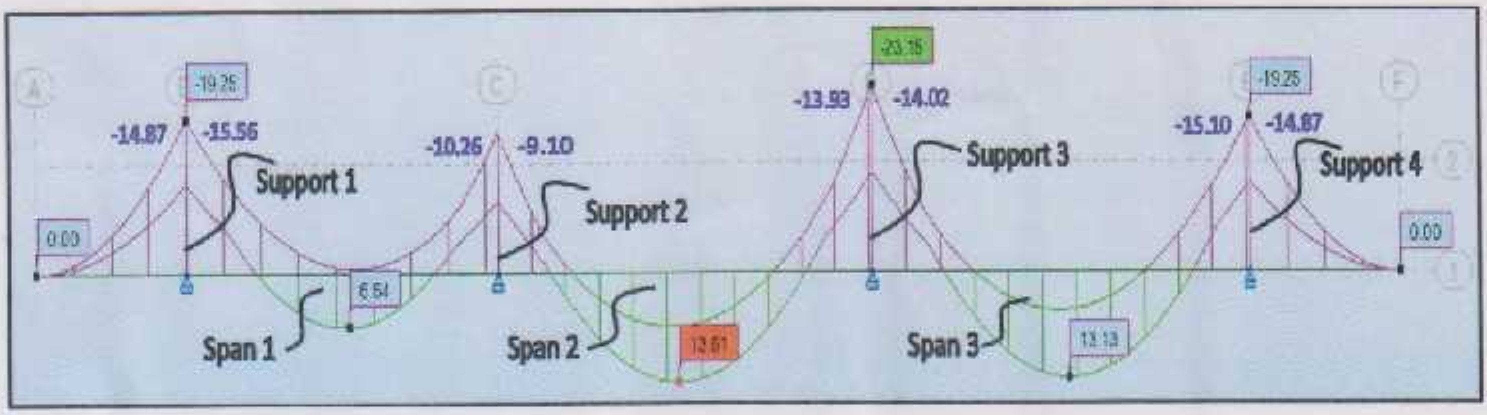


Figure (4-50): **Moment at the face of supports – Rib**

Design of positive moment

From the geometry of T-Section:

bw = 120 mm h= 320 mm hf = t = 80 mm.

The effective width (b_E) according to ACI 8.12.2 b_E is the smallest of :

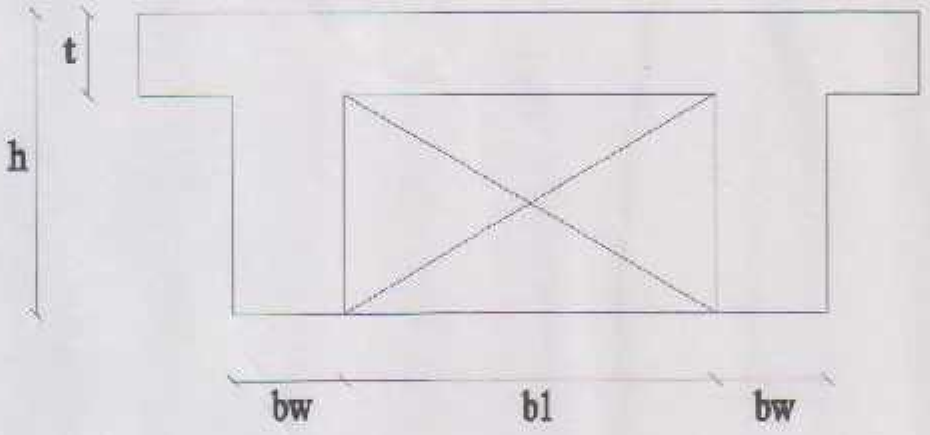
$$b_E \leq \frac{L_B}{4} = \frac{450}{4} = 112.5 \text{ cm}$$

$$b_E \leq b_w + 16\, h_F = 12 + 16 * 8 = 140 \text{ cm}$$

$$b_E \leq b_w + \frac{L_{C1} + L_{C2}}{42} = 12 + \frac{40 + 40}{2} = 52 \text{ cm}$$

Controls .... 52 cm

Requirements for Slab Floor According to *ACI- (318-08)* :



$b_w \geq 10$ cm ACI(8.13.2)

$b_w = 12$ cm ≥ 10 cm

$h \leq 3.5 \times bw = 3.5 \times 12 = 42$ cm ACI(8.13.2)

$h = 32$cm ≤ 42 cm

$b_1 \leq 80$ cm

$b_1 = 40$ cm < 80 cm

$t \geq b_1/12 = 40 / 12 = 3.3$ cm

≥ 5 cm ACI(8.13.6.1)

$t = 8$ cm > 5 cm

Design of span (2)

M_u max = 13.61 KN.m

Check rectangular section or T-section ($a \le t$):

$a = t$

$$\emptyset M_n = 0.9 \times 0.85 \times f_c' \times b_E \times t_f \times \left(d - \frac{t_f}{2}\right)$$

$$\emptyset M_n = 0.9 \times 0.85 \times 24 \times 520 \times 80 \times \left(286 - \frac{80}{2}\right) = 187.89 \text{ KN.m}$$

$$\emptyset M_n = 187.89 \gg 6.64$$

→ $a < t$ for all spans.

The Rib will act as (Rectangular Section).

Design as a rectangular section $b = b_E$

$$Kn = \frac{M_u/\emptyset}{bd^2} = \frac{13.61 * 10^6/0.9}{520 * (286)^2} = 0.356 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 * 24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 * 20.6 * 1.44}{420}}\right) = 0.00085$$

$$As_{req} = \rho\, b\, d = 0.00085 * 520 * 286 = 126.41 \text{ mm}^2$$

$$As_{min} = 0.25 * \frac{\sqrt{f_c}}{f_y} * b * d = 0.25 * \frac{\sqrt{24}}{420} * 120 * 286 = 100.08 \text{ mm}^2$$

Not less than

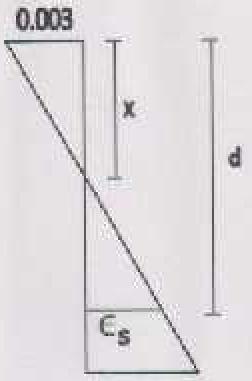
$$As_{min} = \frac{1.4}{f_y} * b * d = \frac{1.4}{420} * 120 * 286 = 114.4 \text{ mm}^2$$

$$As_{req} = 126.41 \text{ mm}^2 > As_{min} = 114.4 \text{ mm}^2$$

$$n = \frac{126.41}{78.54} = 1.6$$

Select 2Ø10 with As = 157.08

Check: strain



$T = C$

$As\, f_y = 0.85\, f_c'\, a\, b$

$157.08 * 420 = 0.85 * 24 * a * 520$

$a = 6.22$ mm

$$x = \frac{6.22}{0.85} = 7.32 \text{ mm}$$

$$\frac{0.003}{x} = \frac{0.003+\epsilon_s}{d}$$

$$\frac{0.003}{7.32} = \frac{0.003+\epsilon_s}{286}$$

$\epsilon_s = 0.114 > 0.005$

So → $\emptyset = 0.9$

Design of span (3)

M_u max = 13.13 KN.m

Design as a rectangular section $b = b_E$

$$Kn = \frac{M_u/\emptyset}{bd^2} = \frac{13.13 * 10^6/0.9}{520 * (286)^2} = 0.343 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 * 24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 * 20.6 * 0.343}{420}}\right) = 0.00082$$

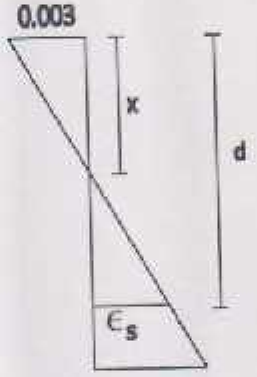
$As_{req} = \rho\, b\, d = 0.00082 * 520 * 286 = 121.95 \text{ mm}^2$

$As_{req} = 121.95 \text{ mm}^2 > As_{min} = 114.4 \text{ mm}^2$

$$n = \frac{121.95}{78.54} = 1.55$$

Select 2Ø10 with As = 157.08

Check: strain



T = C

$As\, f_y = 0.85\, f_c'\, a\, b$

$157.08 * 420 = 0.85 * 24 * a * 520$

a = 6.22 mm

$x = \frac{6.22}{0.85} = 7.32$ mm

$\frac{0.003}{x} = \frac{0.003+\epsilon_s}{d}$

$\frac{0.003}{7.32} = \frac{0.003+\epsilon_s}{286}$

$\epsilon_s = 0.114 > 0.005$

So → $\emptyset = 0.9$

Design of span (1)

M_u max = 6.64 KN.m

Design as a rectangular section $b = b_E$

$$Kn = \frac{M_u/\emptyset}{bd^2} = \frac{6.64 * 10^6/0.9}{520 * (286)^2} = 0.173 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 * 24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 * 20.6 * 0.173}{420}}\right) = 0.00041$$

$As_{req} = \rho\, b\, d = 0.00041 * 520 * 286 = 60.98\ mm^2$

$As_{req} = 60.98\ mm^2 < As_{min} = 114.4\ mm^2$

$As_{req} = 114.4\ mm^2$

$n = \frac{114.4}{78.54} = 1.46$

Select 2Ø10 with As = 157.08

Check: strain

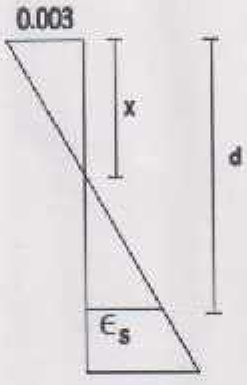
$T = C$

$As\, f_y = 0.85\, f_c'\, a\, b$

$157.08 * 420 = 0.85 * 24 * a * 520$

$a = 6.22 \text{ mm}$

$x = \frac{6.22}{0.85} = 7.32 \text{ mm}$



$\frac{0.003}{x} = \frac{0.003+\epsilon_s}{d}$

$\frac{0.003}{7.32} = \frac{0.003+\epsilon_s}{286}$

$\epsilon_s = 0.114 > 0.005$

So → $\emptyset = 0.9$

Design of negative moment

Design of support (1)

$M_u \max = -15.56$ KN.m

Design as a rectangular section $b_E = b_w$

$Kn = \frac{M_u/\emptyset}{bd^2} = \frac{15.56 * 10^6/0.9}{120 * (286)^2} = 1.761 \text{ Mpa}$

$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 * 24} = 20.6$

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 * 20.6 * 1.761}{420}}\right) = 0.0044$

$As_{req} = \rho\, b\, d = 0.0044 * 120 * 286 = 151 \text{ mm}^2$

$As_{min} = 0.25 * \frac{\sqrt{f_c'}}{f_y} * b * d = 0.25 * \frac{\sqrt{24}}{420} * 120 * 286 = 100.08 \text{ mm}^2$

Not less than

$$As_{min} = \frac{1.4}{f_y} * b * d = \frac{1.4}{420} * 120 * 286 = 114.4\ mm^2$$

$$As_{req} = 151\ mm^2 > As_{min} = 114.4\ mm^2$$

$$n = \frac{151}{78.54} = 1.92$$

Select 2Ø10 with As = 157.08

Check: strain

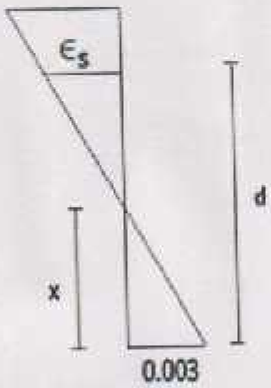
$T = C$

$As\ f_y = 0.85\ f_c'\ a\ b$

$157.08 * 420 = 0.85 * 24 * a * 120$

$a = 26.95$ mm

$$x = \frac{26.95}{0.85} = 31.71\ mm$$



$$\frac{0.003}{x} = \frac{0.003 + \epsilon_s}{d}$$

$$\frac{0.003}{31.71} = \frac{0.003 + \epsilon_s}{286}$$

$\epsilon_s = 0.024 > 0.005$

So → Ø = 0.9

Design of support (2)

M_u max = -10.26 KN.m

Design as a rectangular section $b_E = b_w$

$$Kn = \frac{M_u/\emptyset}{bd^2} = \frac{10.26 * 10^6/0.9}{120 * (286)^2} = 1.161\ Mpa$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 * 24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 * 20.6 * 1.761}{420}}\right) = 0.0029$$

$$As_{req} = \rho\, b\, d = 0.0029 * 120 * 286 = 99.53\ mm^2$$

$As_{req} = 99.53\ mm^2 < As_{min} = 114.4\ mm^2$

$As_{req} = 114.4\ mm^2$

$$n = \frac{114.4}{78.54} = 1.46$$

Select 2Ø10 with As = 157.08

Check: strain

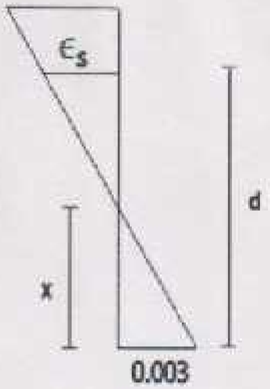
$T = C$

$As\, f_y = 0.85\, f_c'\, a\, b$

$157.08 * 420 = 0.85 * 24 * a * 120$

$a = 26.95\ mm$

$$x = \frac{26.95}{0.85} = 31.71\ mm$$



$$\frac{0.003}{x} = \frac{0.003 + \epsilon_s}{d}$$

$$\frac{0.003}{31.71} = \frac{0.003 + \epsilon_s}{286}$$

$\epsilon_s = 0.024 > 0.005$

So → $\emptyset = 0.9$

Design of support (3)

M_u max = - 14.02 KN.m

Design as a rectangular section $b_E = b_w$

$$Kn = \frac{M_u/\emptyset}{bd^2} = \frac{14.02 * 10^6/0.9}{120 * (286)^2} = 1.587\ Mpa$$

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$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 * 24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 * 20.6 * 1.587}{420}}\right) = 0.0039$$

$$As_{req} = \rho\, b\, d = 0.0039 * 120 * 286 = 133.85\ mm^2$$

$$As_{req} = 133.85\ mm^2 > As_{min} = 114.4\ mm^2$$

$$n = \frac{133.85}{78.54} = 1.7$$

Select 2Ø10 with As = 157.08

Check: strain

$T = C$

$As\, f_y = 0.85\, f_c'\, a\, b$

$157.08 * 420 = 0.85 * 24 * a * 120$

$a = 26.95\ mm$

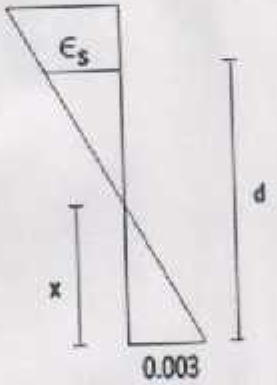
$x = \frac{26.95}{0.85} = 31.71\ mm$

$\frac{0.003}{x} = \frac{0.003 + \epsilon_s}{d}$

$\frac{0.003}{31.71} = \frac{0.003 + \epsilon_s}{286}$

$\epsilon_s = 0.024 > 0.005$

So → $\emptyset = 0.9$



Design of support (4)

M_u max = - 15.1 KN.m

Design as a rectangular section $b_E = b_w$

$$Kn = \frac{M_u/\emptyset}{bd^2} = \frac{15.1 * 10^6/0.9}{120 * (286)^2} = 1.709\ Mpa$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 * 24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 * 20.6 * 1.709}{420}}\right) = 0.0043$$

$As_{req} = \rho\, b\, d = 0.0043 * 120 * 286 = 147.58\ mm^2$

$As_{req} = 147.58\ mm^2 > As_{min} = 114.4\ mm^2$

$$n = \frac{147.58}{78.54} = 1.9$$

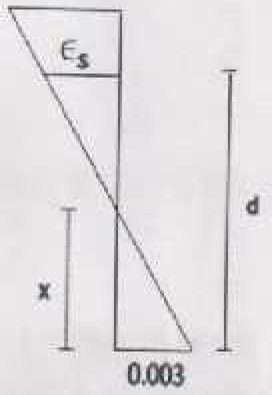
Select 2Ø10 with As = 157.08

Check: strain

$T = C$

$As\, f_y = 0.85\, f_c\, a\, b$

$157.08 * 420 = 0.85 * 24 * a * 120$



$a = 26.95$ mm

$$x = \frac{26.95}{0.85} = 31.71\ mm$$

$$\frac{0.003}{x} = \frac{0.003 + \epsilon_s}{d}$$

$$\frac{0.003}{31.71} = \frac{0.003 + \epsilon_s}{286}$$

$\epsilon_s = 0.024 > 0.005$

So → $\emptyset = 0.9$

4.2.2 Design of mat foundation (1)

When the bearing capacity of soil is low ,isolated footing and strip footing under shear wall are released by a raft foundation. In such case ,a solid reinforced concrete rigid slab constructed under the entire building. Structurally ,raft foundations resisting directly on soil act as flat slab or flat plate, upside down, loaded upward by bearing pressure and downward by the concentrated column reaction and distributed shear wall reaction, the advantage of raft foundation is that help in reducing different settlement of individual columns relative to each other, which might be caused by local variations in the quality of subsoil, or other causes.

The design of raft foundations may be carried out by one of two methods:

- The conventional rigid method and;
- The finite element method utilizing computer programs.

The conventional method is easy to apply and the computations can be carried out using hand calculation. However ,the application of conventional method is limited to rafts with regular arrangement of columns only.

In contrast, the finite element method can be used for analysis of raft regardless of the column arrangement, loading conditions, and existence of cores and shear walls, commercially available computer programs can be used. The user should ,however, have sufficient background and experience.

4.2.2.1 Rigidity of foundation

The rigidity of mat foundation calculated as the following :

Rigidity of raft foundation (B = 6.5 m * L = 7.7 m) :

$Ix = \frac{bh^3}{12}$, $Ix = \frac{6.5 \times 7.7^3}{12} = 247.3\ m^4$ – Control

$Iy = \frac{bh^3}{12}$, $Iy = \frac{7.7 \times 6.5^3}{12} = 176.3\ m^4$

$$K = \frac{I \times Ec}{Ec \times B^3}$$

I : moment inertia of raft foundation material

Ec: modules elasticity of concrete =4700 × $\sqrt{fc}$

B :width of mat foundation

Es : modules elasticity of soil

Ec=4700 × $\sqrt{24}$=23025.20 MN/m2

Es (soft rock) =1000 MN/m2 from *"table (6-12) principle of foundation"*

$$K = \frac{247.3 \times 23025.2}{1000 \times 7.6^3} = 12.97 >> 0.5 \quad \text{Rigid}$$

As the value of K > 0.5 it should be design as rigid mat foundation, but there no software design mat as rigid ,more over its difficult to design it by hand manual ,so we consider it as flexible and this method gives result close to the result of rigid method

4.2.2.2 Structural analysis of mat

Analysis result:

Safe is the software used to analyze the mat as whole , the program use the strip element method in finding the internal forces for the mat.

The program consider the mat as a tow way slab so that we get internal forces in both vertical and horizontal directions . A screen capture for the model is shown mat strips in both direction

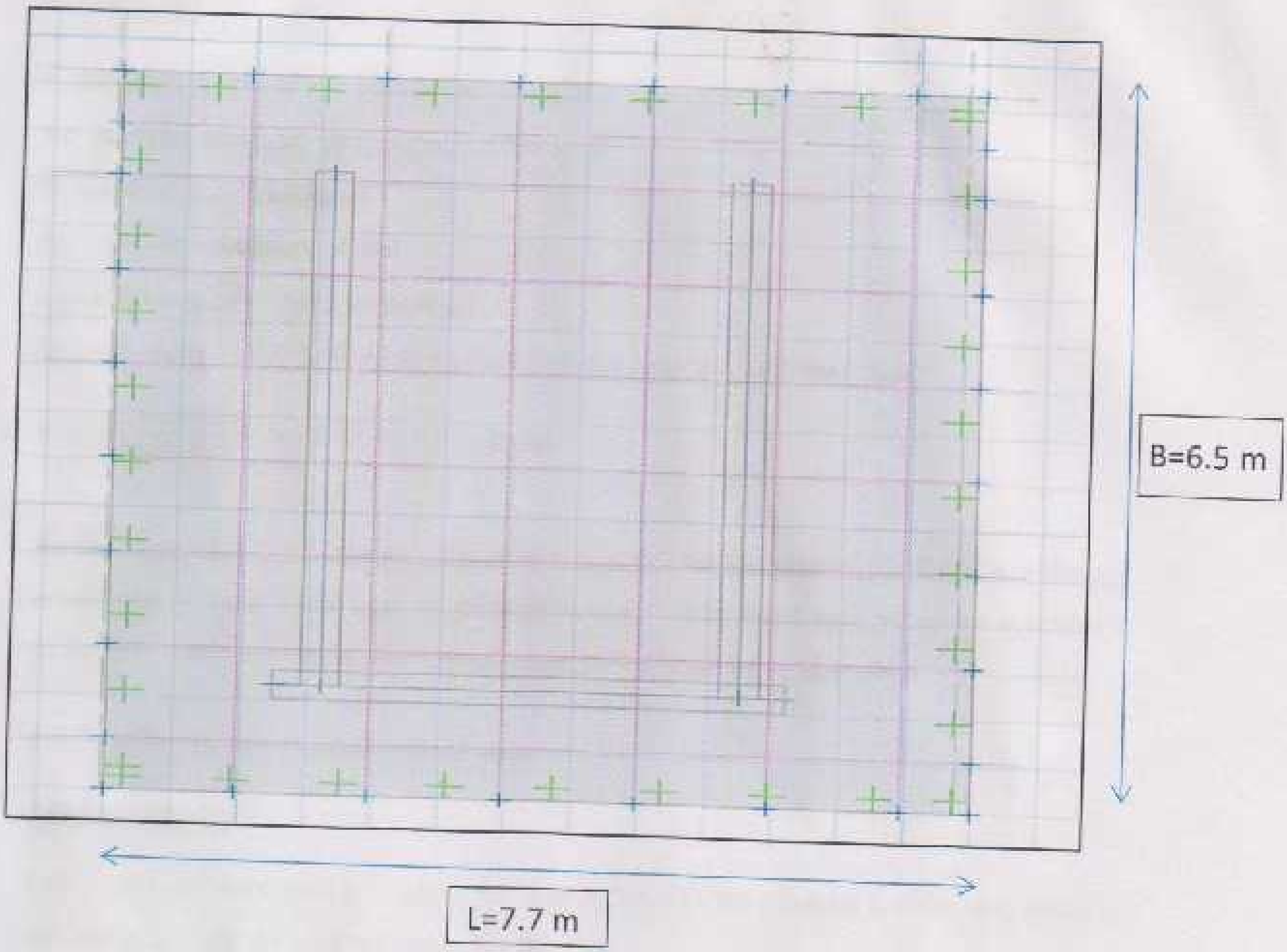


Figure (4-51): Mat strips

Soil Reaction

The maximum soil reaction on the mat due to service load is 220 (KN/m^2) is less than the allowable bearing capacity of the corresponding layer 235 (KN/m^2).

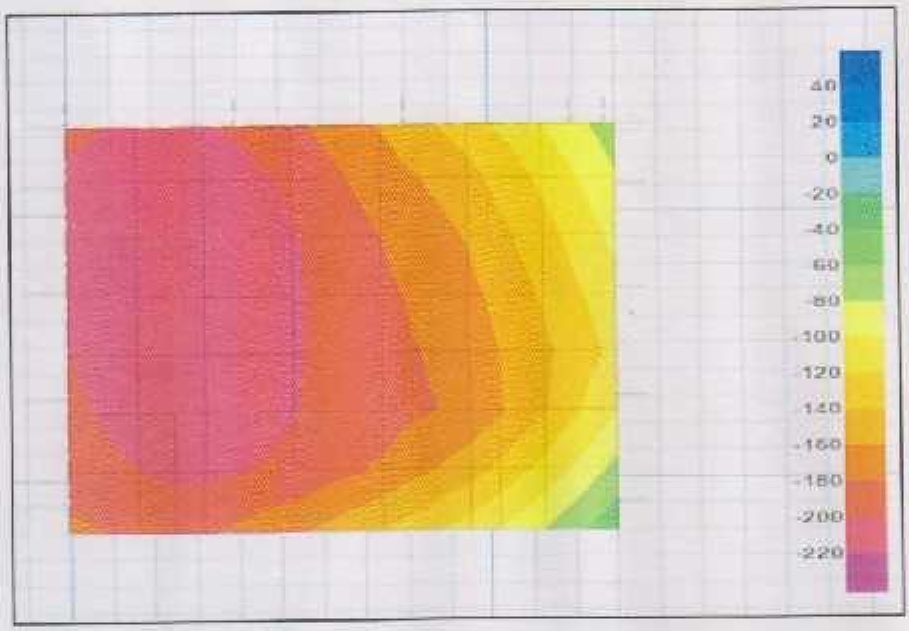


Figure (4-52): Soil reaction of mat

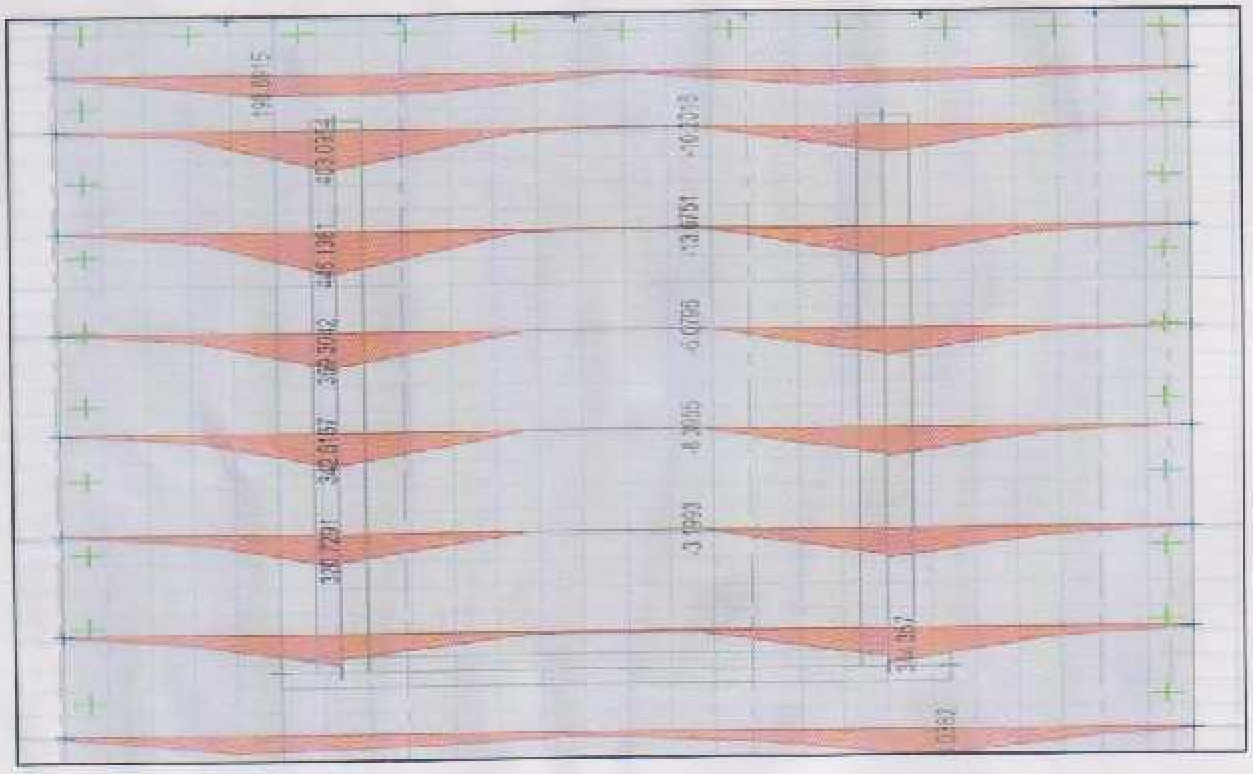


Figure (4-53): Bending moment in x direction

Mxx : which is used to determine the vertical reinforcement steel

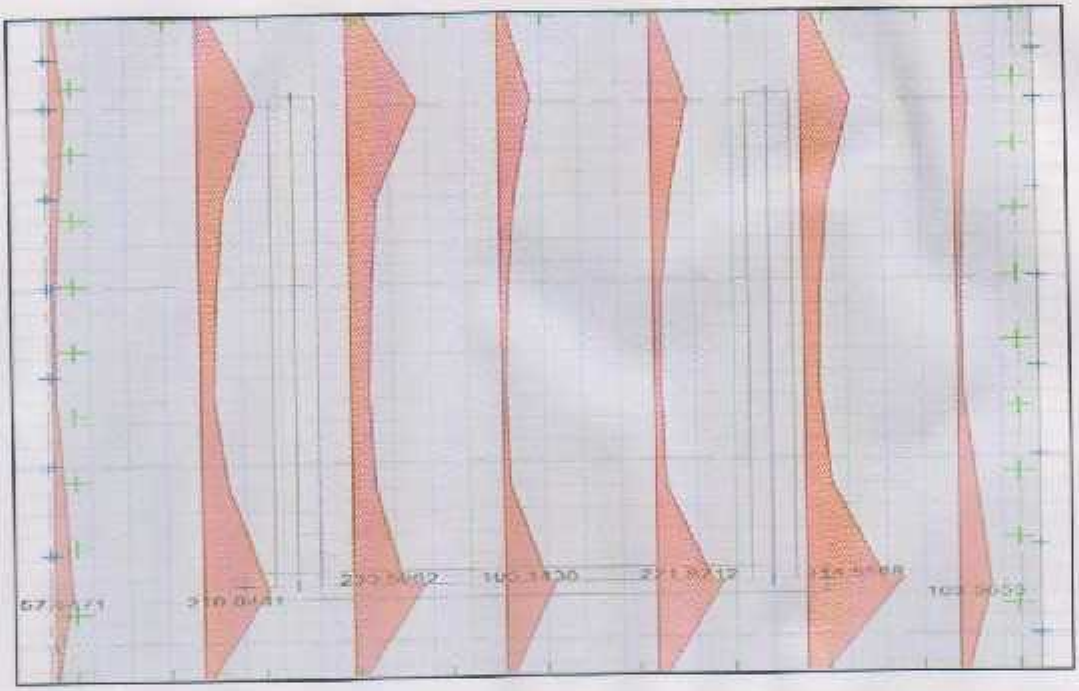


Figure (4-54): Bending moment in y direction

Myy : which is used to determine the horizontal reinforcement steel

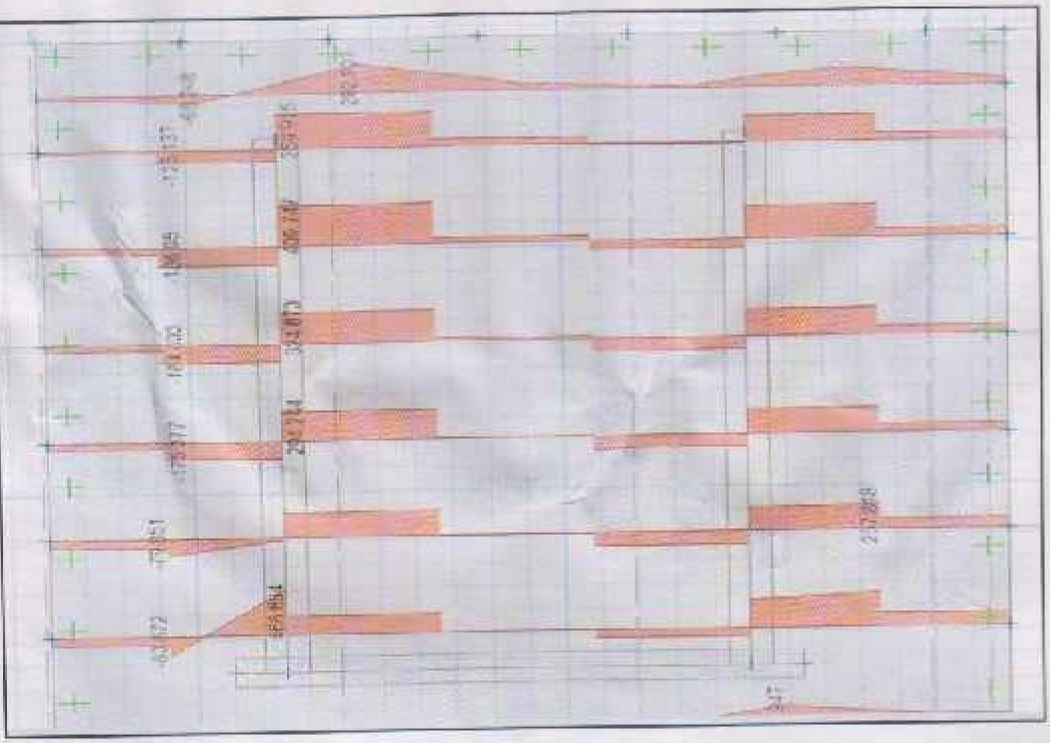


Figure (4-55): Shear force in x direction

Vxx :Shear force in the vertical direction

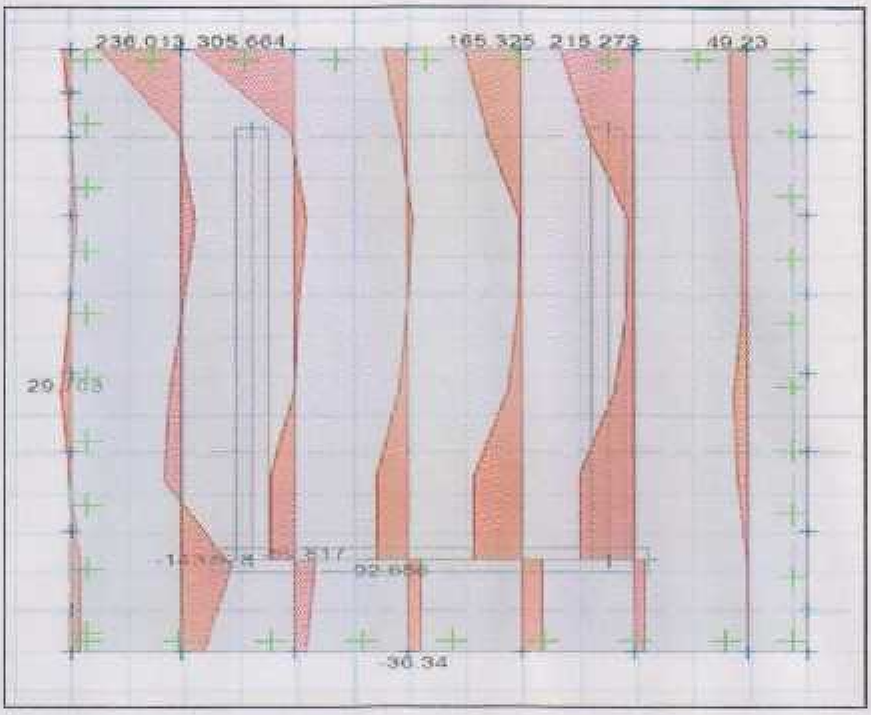


Figure (4-56): Shear force in y direction

Vyy :Shear force in the vertical direction

Table (4-2): Envelope Results Table (mat1) : Maximum internal forces

Mxx	Myy	Vxx	Vyy
182 KN.m	192 KN.m	410 KN	306 KN

4.2.2.3 Design of shear force

$Vu_{max} = 410$ KN/m

For 1 meter strip :-

Thickness t =750 cm , d=750-50-16 =684 mm.

$\emptyset \times Vc = 1/6 \times 0.75 \times \sqrt{f_{c'}} \times b_w \times d$

$\emptyset \times Vc = 1/6 \times 0.75 \times \sqrt{24} \times 1000 \times 684 = 418.862$ KN/m > 410 KN/m , $O.K$

$\emptyset \times Vc > Vu$ Thikness is safe & no shear rainforcement is required

4.2.2.4 Design of bending moment

Design of bottom reinforcement in x direction:

Mxx=182 KN.m

$$Kn = \frac{\frac{M_u}{\emptyset}}{bd^2} = \frac{182 \times \frac{10^6}{0.9}}{1000 \times (684)^2} = 0.4322 \text{ Mpa}$$

$$m = \frac{f_y}{0.85f_c'} = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho_{required} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right)$$

$$\rho_{required} = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 * 0.432 * 20.6}{420}}\right) = 0.00104$$

$As_{req} = \rho_{req} \times b \times d$

$As_{req} = 0.0010 \times 1000 \times 684 = 684\ mm^2/m$

$As_{min} = \rho_{min} \times b \times h \qquad ,(eq - 7.12.2.1\ ACI\ 318 - 08)$

$As_{min} = 0.0018 \times 1000 \times 750 = 1350\ mm^2/m > As_{req} \rightarrow$ control

Select $\emptyset 16@14cm$ with $As = 1436\ mm^2/m$

<u>check maximum spacing :</u>

$$s = 380\left(\frac{280}{f_s}\right) - 2.5c_c = s = 380\left(\frac{280}{280}\right) - 2.5 \times 50 = 25.5 > 18\ \text{cm ok}$$

<u>Design of bottom reinforcement in y direction:</u>

Myy=192 KN.m

$$Kn = \frac{\frac{M_u}{\emptyset}}{bd^2} = \frac{192 \times \frac{10^6}{0.}}{1000 \times (684)^2} = 0.456\ \text{Mpa}$$

$$m = \frac{f_y}{0.85f_c'} = \frac{420}{0.85 \times 24} = 20.6$$

$As_{req} = \rho_{req} \times b \times d$

$$\rho_{required} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right)$$

$$\rho_{required} = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 * 0.456 * 20.6}{420}}\right) = 0.0011$$

$As_{req} = 0.0011*1000*684 = 753\ mm^2/m$

$As_{min} = \rho_{min} \times b \times h$

$As_{min} = 0.0018*1000*750 = 1350\ mm^2/m > As_{req} \rightarrow$ control

Select $\emptyset 16@14cm\ with\ As = 1436mm^2/m$

check maximum spacing :

$$s = 380\left(\frac{280}{f_s}\right) - 2.5c_c = s = 380\left(\frac{280}{280}\right) - 2.5 \times 50 = 25.5 > 18 \text{ cm ok}$$

Design of top reinforcement in both direction:

$My_{max} = 56$ KN.M < 192 KN.M

$As_{req} = As_{min} = 0.0018*1000*750=1350\ mm^2/m$

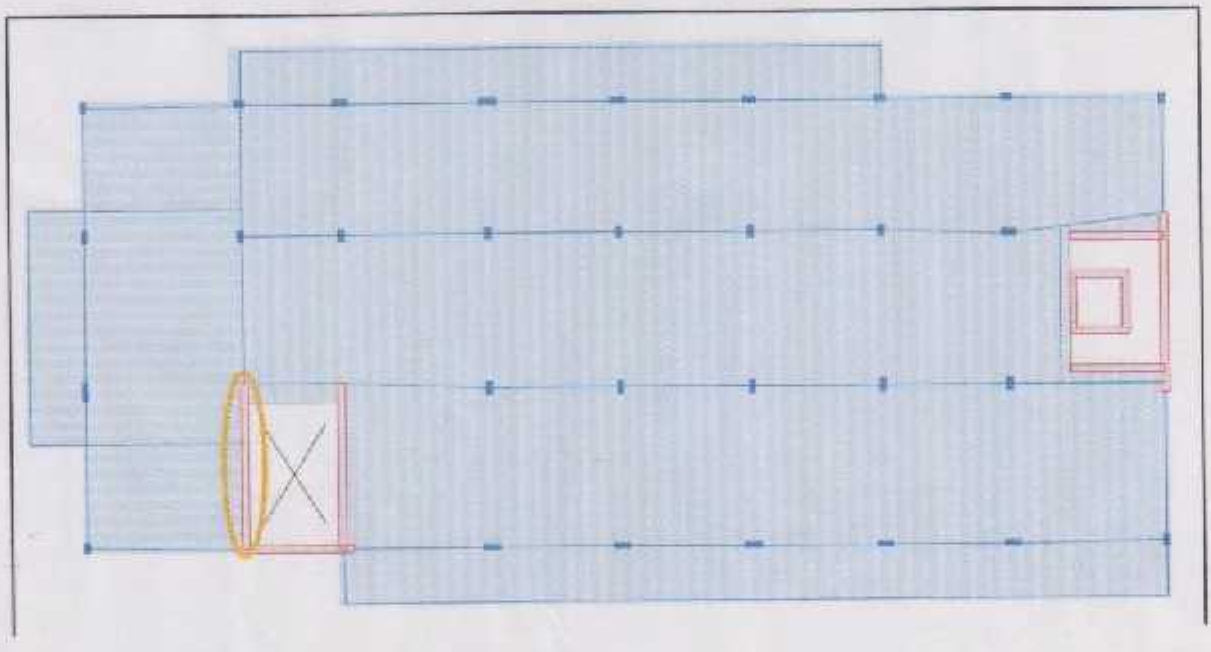
Select $\emptyset 18@18cm\ with\ As = 1411\ mm^2/m$

check maximum spacing :

$$s = 380\left(\frac{280}{f_s}\right) - 2.5c_c = s = 380\left(\frac{280}{280}\right) - 2.5 \times 50 = 25.5 > 18 \text{ cm ok}$$

4.2.3 Design of shear wall (SW1)

4.2.3.1 Structural Analysis of shear wall



Lw = 5.475 m hw = 17.95 m b = 250 cm

Lw < hw , d=0.8*Lw

d=0.8*5.475 , d=4.38m

Critical section for concrete is the smallest of:

Lw / 2 = 5.475 / 2 = 2.735 m

hw / 2 = 17.95 / 2 = 8.975 m

Story height =2.1 m

Mu_1 at critical section = 6774 KN.m

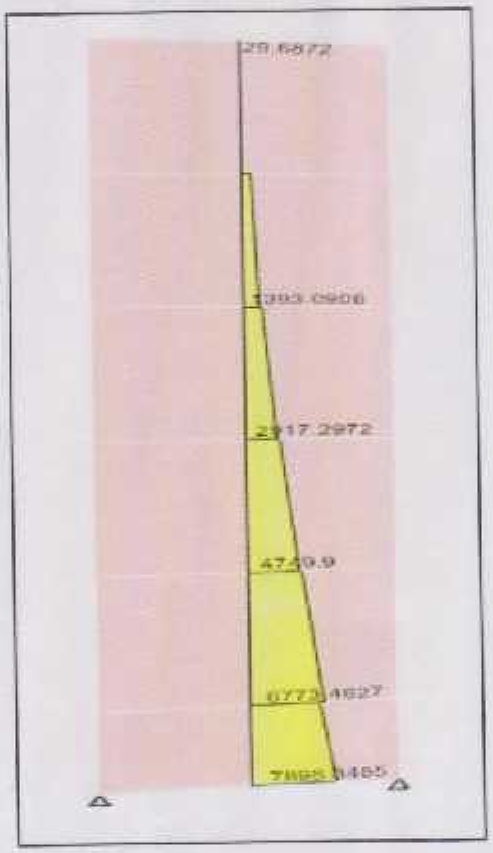


Figure (4-57): Bending moment about strong axis

M_{33} : the which is resisted by the uniform vertical and boundary steel.

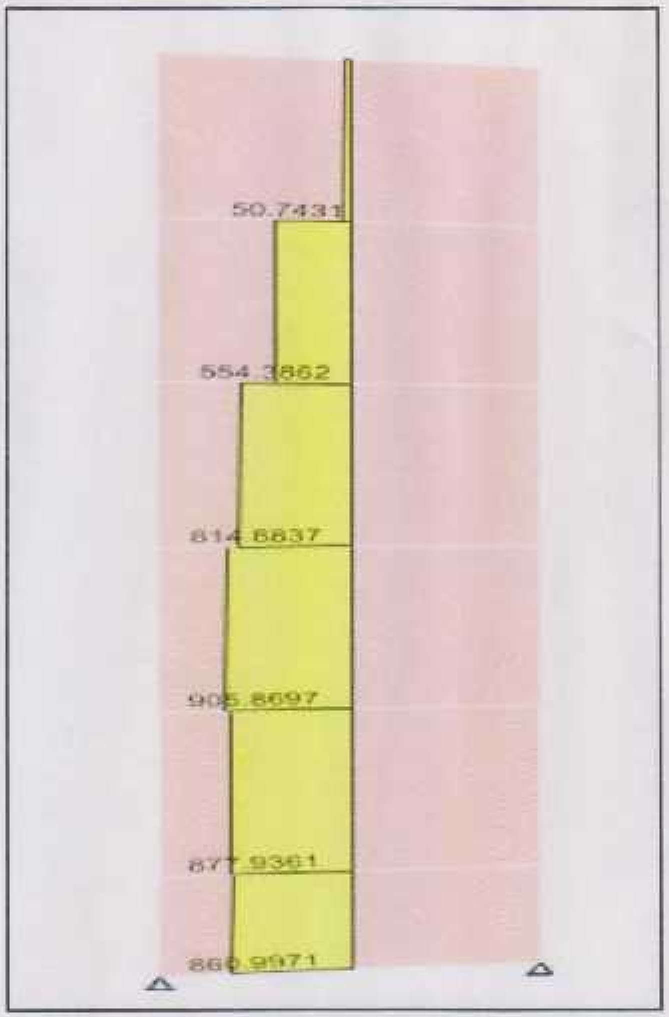


Figure (4-58): Shear force about strong axis

V_{22} : the shear about strong axis which is resisted by the uniform horizontal steel.

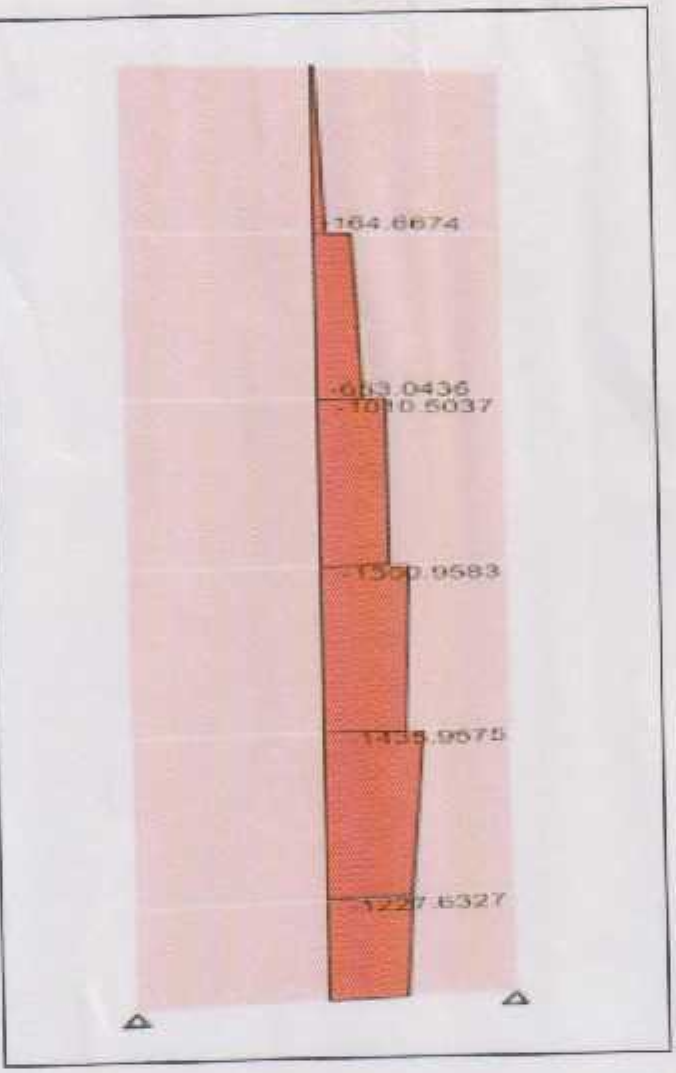


Figure (4-59): Axial force in shear wall

Pu : the axial force which is resisted by the uniform vertical steel.

Table (4-3): Envelope Results Table (SW1) : Maximum internal forces

M_{33}	Vu	Pu
7899 KN.m	906 KN.m	1436 KN

4.2.3.2 Design of horizontal reinforcement

Vc is the smallest of :

$Vc = \frac{1}{6} \times \sqrt{f_c'} \times h \times d$, $eq. 11.2.1.1\ ACI - 318 - 08$

$Vc = \frac{1}{6} \times \sqrt{24} \times 250 \times 4380 = 895$ KN

$Vc = 0.25 \times \sqrt{f_c'} \times h \times d + \frac{N_u \times d}{4 l_w}$ $eq. 11.9.6\ ACI\ 318 - 08$

$Vc = 0.25 \times \sqrt{24} \times 300 \times 4380 + \frac{1436 \times 4380}{5475} = 1342$ KN

$$Vc = \left[0.05 \times \sqrt{f_c'} + \frac{L_w \left(0.1 \times \sqrt{f_c'} + 0.2 \times \frac{N_u}{h \times l_w}\right)}{\frac{M_u}{V_u} - \frac{L_w}{2}}\right] h \times d \quad eq. 11.9.6\ ACI\ 318 - 08$$

$$Vc = \left[0.05 \times \sqrt{24} + \frac{5.475\left(0.1 \times \sqrt{24} + 0.2 \times \frac{1456}{250 \times 4380}\right)}{\frac{6774}{906} - \frac{5.475}{2}}\right] 250 \times 4380 = 889.3 \text{ KN}$$

Vc = 1065.821 , controls

$\emptyset \times Vc = 0.75 \times 1065.821 = 800$ KN

$\emptyset \times$ Vc<Vu shear reinforcement required

$Vs = Vn - Vc = \frac{V_u}{\emptyset} - Vc = \frac{906}{0.75} - 889.3 = 117.2$ KN.

$$\frac{A_{hs}}{s} = \frac{A_{hs}}{f_y \times d} = \frac{142.2 \times 10^3}{420 \times 4380} = 0.065 \frac{mm^2}{mm}$$

$\frac{A_{hs}}{s} \min = 0.0025 \times h$

$\frac{A_{hs}}{s} \min = 0.0025 * 250 = 0.625 \frac{mm^2}{mm}$ take Ø10 with As=158 mm2

$S = \frac{157}{0.625} = 251.2$ mm

Select Ø10 @200 mm in each layer < S_{max}

S=3*h=3*250=750 mm

S=450 mm control

$S=\frac{Lw}{5}=\frac{5.475}{5}=1095$ mm

4.2.3.3 Design of uniform vertical reinforcement

$$\frac{\sum h_w}{L_w}=\frac{5\times 3.47+2.1}{5.475}=3.55\text{m.}$$

$\frac{A_{sv}}{s}\geq(0.0025+0.5\left(2.5-\frac{\sum h_w}{L_w}\right)(\frac{Ah}{b\times s}-0.0025))\times h\leq 2.5$, eq. 11.30 ACI 318 – 08

$\frac{A_{sv}}{s}=0.0025+0.5\left(2.5-\frac{19.45}{5.475}\right)(\frac{158}{250\times 250}-0.0025)\times 250=0.625\ \frac{mm^2}{mm}$

take Ø10 with As=158 mm2

$S=\frac{157}{0.625}=200$ mm

Select Ø10 @200 mm in each layer < Smax

S = 3*h=3*250=750 mm

S = 450 mm control

$S=\frac{Lw}{3}=\frac{5.475}{3}=1825$ mm

4.2.3.4 Design of bending moment

Check moment strength based on required uniform vertical reinforcement for shear.

$A_{st}=\frac{5475}{200}\times 2\times 78.5=4298\ mm^2$

$$\frac{Z}{Lw} = \frac{1}{2 + \frac{0.85 \times f_c' \times B \times Lw \times h}{Asv \times fy}}$$

$$\frac{Z}{Lw} = \frac{1}{2 + \frac{0.85 \times 24 \times .85 \times 5475 \times 250}{4298 \times 420}}$$

$$M_{uv} = 0.9 \times (0.5 \times A_{sv} \times fy \times Lw \times (1 - \frac{Z}{2 \times Lw}))$$

$$M_{uv} = 0.9 \times (0.5 \times 4298 \times 420 \times 5475 \times (1 - 0.5 \times 0.056)) = 4323 \text{ KN.m}$$

$M_{uv} = 4323 < M_u = 7899$ KN.m , $\therefore$ boundary steel is required

$M_{UB} = M_U - M_{UV}$, $M_{UB} = 7899 - 4323 = 3576$ KN.m

$X \geq \frac{Lw}{600 \times \frac{\delta u}{hw}}$, *Sec 21.9.6.2 ACI 318 – 08*

$(\frac{\delta u}{hw})_{min} = 0.007$, *Sec 21.9.6.2 ACI 318 – 08*

$X_{max} = \frac{5475}{600 \times 0.007} = 1304$ mm

Boundary length $LB \geq \frac{X}{2}$

$LB = \frac{1304}{2} = 652\ mm$

$LB \geq X - 0.1 \times L_w$

$LB = 1304 - 0.1 \times 5475 = 757$ mm

Select LB = 800 mm > 757 mm

$$A_{SB} = \frac{Mub/_{0.9}}{fy \times (Lw - LB)}$$

$$A_{SB\ req} = \frac{3576 \times 10^6/_{0.9}}{420 \times (5475 - 800)} = 2024\ mm2$$

Select 14 Ø14 with As=2155 $mm^2 > 2024\ mm2$

4.2.4 Design of stair (ST01)

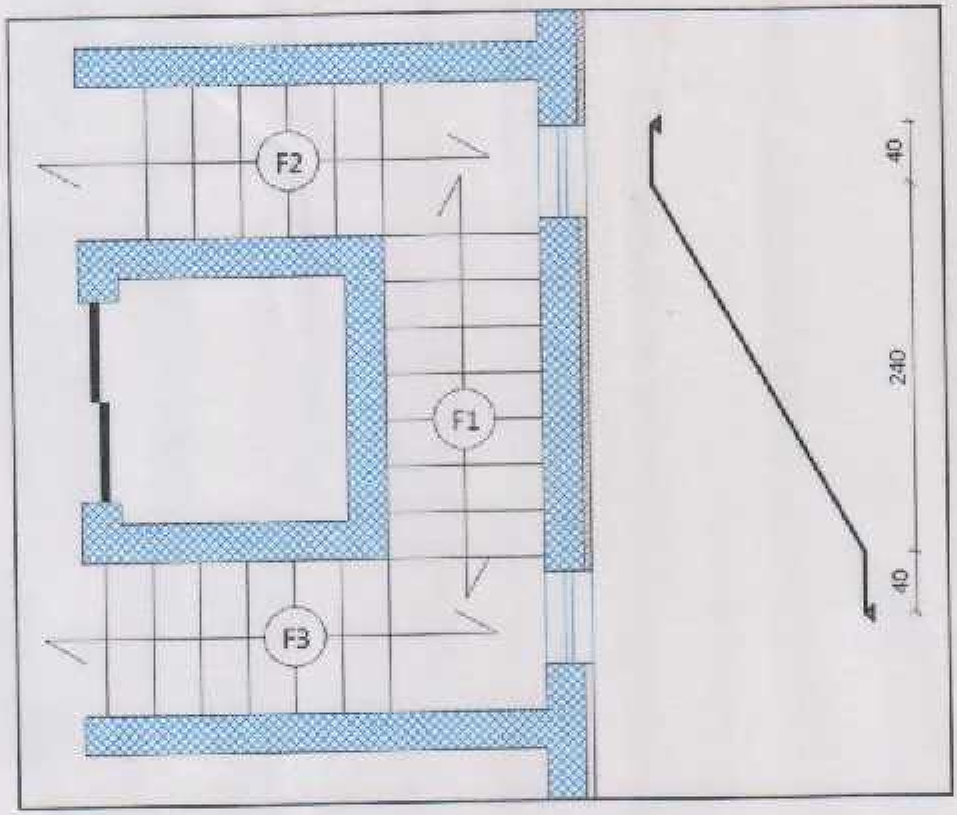


Figure (4-60): Flight F1 system

Design of flight (1)

- Limitation of deflection:

$h \geq h_{min}$

$$h = \frac{L}{20} = \frac{3.2}{20} = 16 \text{ cm}$$

Select $h = 20 \text{ cm} \geq h_{min}$

Inclination angle = 29.54°

Loads

$$\text{Slab} = 0.2 \times 25 \times \frac{1}{\cos 29.54} = 5.74 \text{ Kn/m}$$

$$\text{Vertical Morter} = 0.03 \times 0.17 \times 22 \times \frac{1}{0.3} = 0.374 \text{ Kn/m}$$

$$\text{Plaster} = 0.03 \times 22 \times \frac{1}{\cos 29.54} = 0.758 \text{ Kn/m}$$

$$\text{Vertical Tiles} = 0.03 \times 0.17 \times 23 \times \frac{1}{0.3} = 0.391 \text{ Kn/m}$$

$$\text{Horizontal Tiles} = 0.04 \times 1 \times 23 \times \frac{33}{30} = 1.012 \text{ Kn/m}$$

$$\text{Horizontal Morter} = 0.03 \times 1 \times 22 = 0.66 \text{ Kn/m}$$

$$\text{Triangles} = 0.17 \times \frac{25}{2} \times 1 = 2.125 \text{ Kn/m}$$

Total Dead load = 11.06 Kn/m

Live Load = 4 Kn/m

$$qu = 1.2 \times 11.06 + 1.6 \times 4 = 19.672 \text{ Kn/m}$$

Analysis

Design of Shear Strength

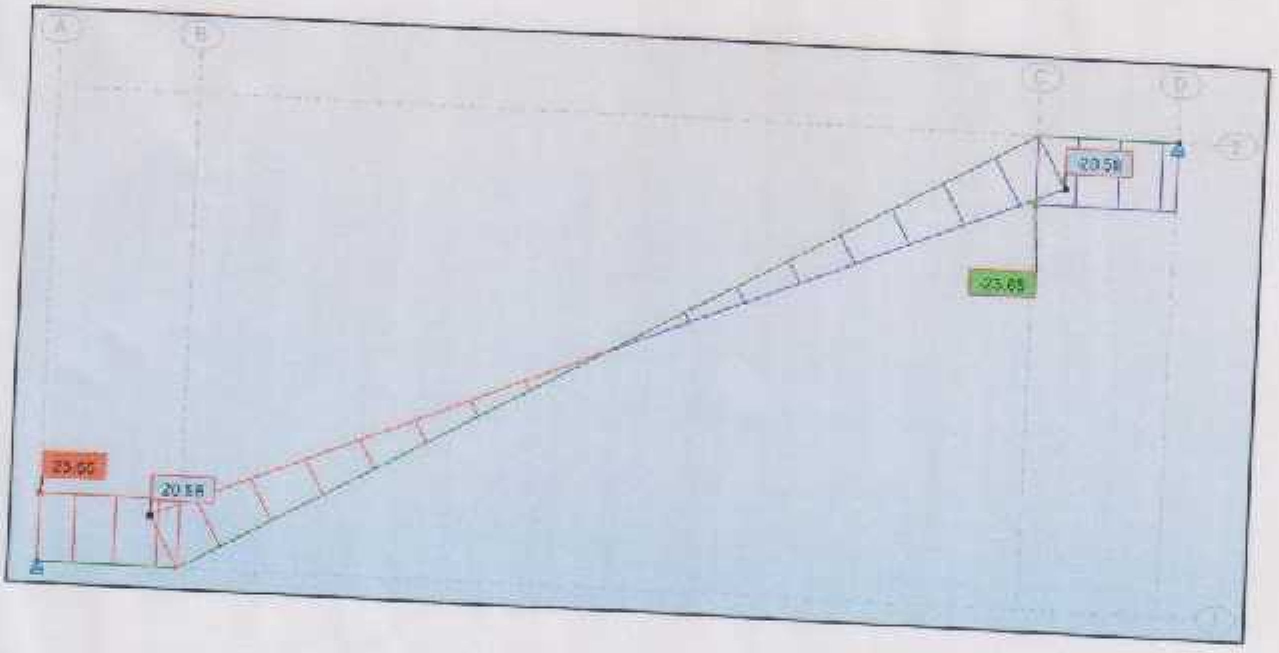


Figure (4-61): **Flight Shear Force Diagram**

Vu = 20.85 KN

$Vc = \frac{1}{6}\sqrt{f_c'} b_w d = \frac{1}{6}\sqrt{24} \times 1000 \times 174 \times 10^{-3} = 142.07$ KN , ACI (11-3)

$\emptyset$ Vc = 0.75×142.07 = 106.55 KN > 18.04 KN

No Shear Reinforcement is required.

Design of Bending Moment

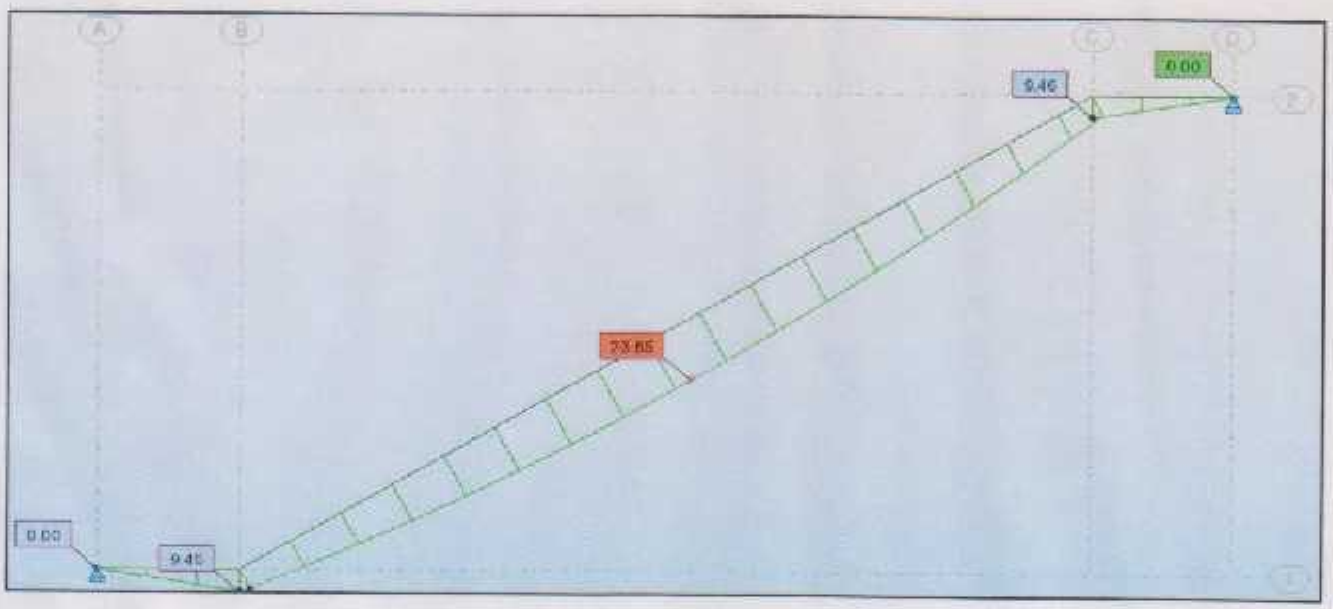


Figure (4-62): **Flight Moment Diagram**

Mu Max = 23.65 KN.M

$d = 200 - 60 = 174$ mm

$$Kn = \frac{M_u/\emptyset}{bd^2} = \frac{23.65 * 10^6/0.9}{1000 * (174)^2} = 0.868 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 * 24} = 20.59$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{20.59}\left(1 - \sqrt{1 - \frac{2 \times 0.868 \times 20.59}{420}}\right) = 0.002$$

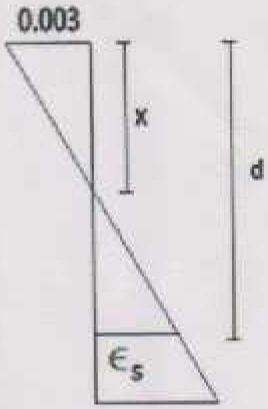
$As_{req} = \rho\, b\, d = 0.002 * 1000 * 174 = 348 \text{ mm}^2$

$As_{min} = 0.0018\, b\, h = 0.0018 * 1000 * 200 = 360 \text{ mm}^2$, ACI-318-08 (7.12.2.1)

$As_{req} = As_{min} = 360 \text{ mm}^2$

<u>**Select Ø10/20cm ,**</u> with As = ($\pi \times 10^2$ / 4) × 100/20 = 392.5 mm²

Check: strain



$T = C$

$As\, f_y = 0.85\, f_c'\, a\, b$

$392.5 * 420 = 0.85 * 24 * a * 1000$

$a = 8.08 \text{ mm} \quad x = \dfrac{8.08}{0.85} = 9.5 \text{ mm}$

$$\frac{0.003}{x} = \frac{0.003 + \epsilon_s}{d}$$

$$\frac{0.003}{9.5} = \frac{0.003 + \epsilon_s}{174}$$

$\epsilon_s = 0.052 > 0.005$

OK → $\emptyset = 0.9$

Secondary Reinforcement

$As_{req} = As_{min} = 360 \text{ mm}^2$

Select Ø10/20cm

4.3 Design of Swimming Pool

4.3.1 Background theory

Due to the internal pressure of the liquid stored in such structures, the walls and floors are mainly subject to tensile forces, bending moments and eccentric tension which cause in most cases critical tensile stresses on the surface of the different elements facing the liquid. If such elements are designed according to the general principles adopted in ordinary reinforced concrete, cracks will be developed and the liquid contained in the tank has "the possibility to penetrate under its hydros-tatic pressure through the cracks and cause rusting of the steel reinforcement. Therefore, special provisions must be taken to prevent the formation of such cracks. Such provisions generally lead to an increased thickness of the walls towards their foot and at their other corners. If the effect of this increase is not considered, it may lead to serious defects so that a thorough investigation is absolutely essential.

Sections of liquid containers must be so designed that no cracks in concrete are allowed in the fibers facing the liquid, because if such cracks are allowed, the liquid in the container will penetrate through these cracks and cause rusting of the steel reinforcement which must be prevented by all possible means.

In order to satisfy this requirement, the concrete dimensions must be chosen so that the tensile stresses in concrete- if they take place on the liquid side are smaller than its tensile strength. But as the tensile strength of concrete cannot be guaranteed , the tension steel rein-forcement must be designed to carry all the tensile stresses i.e. the concrete in tension is neglected.

Design consideration:

the safe design of water retaining structure must have :

a) Adequate resistance against cracking and

b) Adequqte strength.

Control of cracking

crack width must be minimize in swimming pool wall to prevent leakage and corrosion of reinforcement . A criterion for flexural crack width is provided in **ACI318-11 (section10.6.4) .**

Flexural tension reinforcement shall be well distributed within maximum flexural tension zones of a member cross section as required by **10.6.4.**

10.6.4— The spacing of reinforcement closest to the tension face, **s**, shall not exceed that given by equation (10-4)

$$s = 380\left(\frac{280}{f_s}\right) - 2.5c_c$$

but not greater than **300(280/fs)**, where c_c is the least distance from surface of reinforcement or prestressing steel to the tension face. If there is only one bar or wire nearest to the extreme tension face, s , used in Eq. **(10-4)** is the width of the extreme tension face.

Calculated stress **fs** in reinforcement closest to the tension face at service load shall be computed based on the un factored moment. It shall be permitted to take **fs** as **2/3fy** .

4.3.2 Loading conditions

The swimming pool must be designed to withstand the loads that it will be subjected to during many years of use. But it is equally important to consider loads during constructions .these conditions are:

case 1: the swimming pool is full of water during leakage test period and prior to backfill.
case 2: backfill the swimming pool and no water is in the pool.

According to ACI 350, the proper design of tank will include the full effect of the soil loads and water pressure without taking into account loads acting in directions that minimize the effects of each other.

Overload Factors

according to ACI 318-08 the load factor to be used for lateral liquid pressure(F) is 1.2 and for the lateral soil pressure (H) is 1.6 ,given in equation (9-2) :

$$U = 1.2(D + F) + 1.6(L + H)$$

Thicknesses of Walls and Base Sections

The minimum thickness for walls that are greater than or equal to (3 m) in height is (300 mm) **(ACI 350-01, Section 14.6.2).** For lower walls, (250 mm) is the practical minimum thickness for walls with reinforcement in each face. Greater thicknesses are desirable where water-stops are used, due to the limited space available for both reinforcement and water-stop placement with adequate concrete cover. For walls greater than (3 m) in height, a reasonable rule of thumb is to use a minimum wall thickness equal to 1/12 the wall height for cantilever walls and 1/16 the height for propped cantilever walls. The minimum thickness of footings and mat foundations should generally be (300 mm). Depending on the previous provision we select the thickness for wall with 6 m in height to be 50 cm and for the 3m in height to be 30 cm and for the base slab also to be 30 cm.

4.3.3 Calculation of loads

4.3.3.1 Loads on Walls:

Case 1 : the pool is filled with water and no backfill behind the walls

The hydrostatic pressure on the walls equals to the density of water (9.81 KN/m^3) multiplied by the height of water (H)

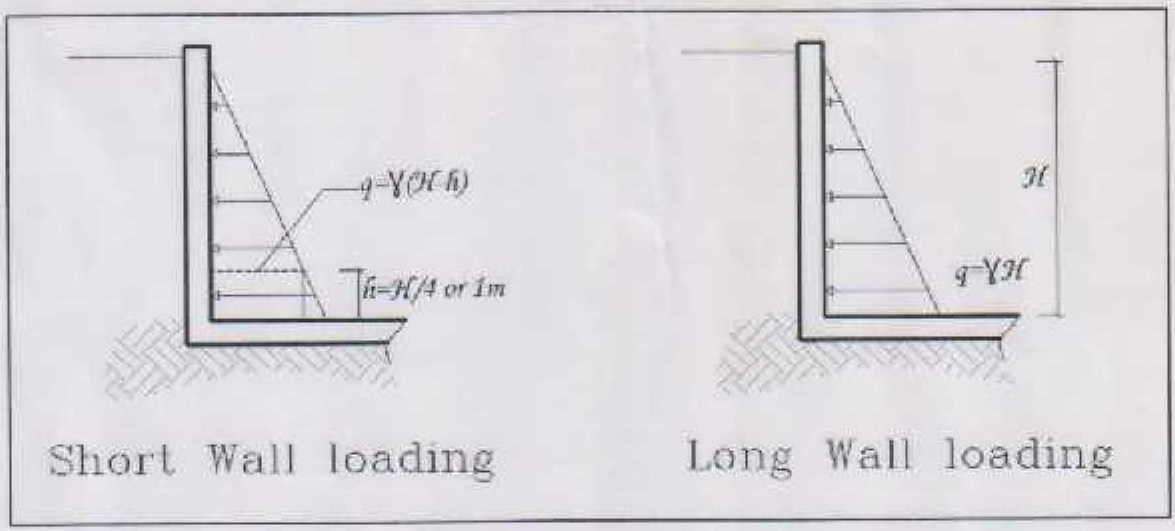


Figure (4-63): Loads on wall due to water pressure

Case 2: Backfill is behind the walls and the Pool is empty.

The loads on the wall come from soil action and surcharge onto the upper ground surface that is taken as (5 KN/m^2).

For the simplicity sake and because the second layer is 6 m thick , the soil pressure will be calculated from the second layer only producing a linear triangle load:

a) loads on long walls:

The active earth pressure is calculated as :$\boldsymbol{k} = \frac{1-\sin\emptyset}{1+\sin\emptyset}$

Then (K) multiplied by the height and the density of soil $\boldsymbol{q} = \boldsymbol{k\gamma H}$ for the pressure.

The surcharge is calculated as: $\boldsymbol{w} = \boldsymbol{kP}$

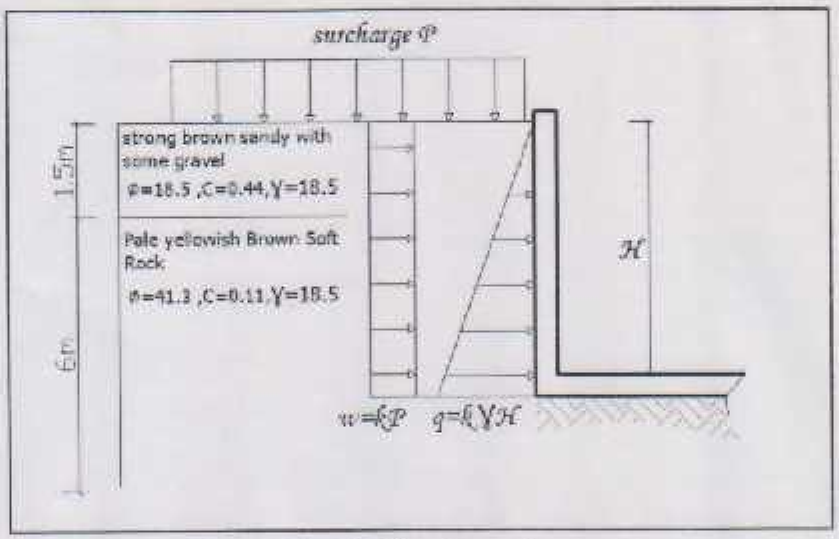


Figure (4-64): Loads on long walls

b) loads on short walls:

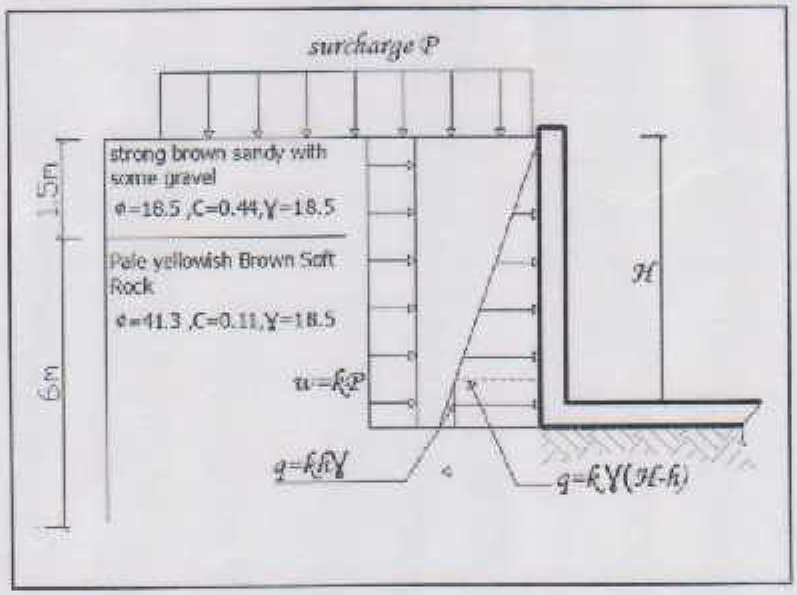


Figure (4-65): Loads on short walls

4.3.3.2 Loads on Base Slabs

Case 1: only case one is applied when the pool is filled with water.

The hydrostatic pressure on the Base slabs equals to the density of water (9.81 KN/m^3) multiplied by the height of water (H).

Due to variety of levels of the pool different subgrades modulus (Kz) appears under the base slabs. The following table shows that values according to the soil report (the recommended value by the report is used).

Table (4-4): **Subgrade modulus of the base slabs**

Slab level (m)	q allowable (Kg/cm^{-2})	q allowable (KN/m^2)	Kz (KN/m^3)
-1.5	2.4	235.44	28252.8
-3	2.4	235.44	28252.8
-6	1.9	186.39	22366.8

The subgrade modulus is calculated for 25 mm allowable settlement and safety factor of 3 as :

$$Kz = \left(\frac{1}{\frac{0.025}{3}}\right) \times q_{allowable}$$

$$Kz = 120 \times q_{allowable}$$

4.3.4 Structural Analysis of the pool walls

Since the exact analysis of rectangular swimming pool is very much complicated . An approximate approach for the design of rectangular tanks is introduced herein .

However, the analysis of the Pool walls and base will be carried out using the computer software , since there are many cases and loading conditions to be considered and the accuracy that the program gives .

<u>Approximate Method for Rectangular Tanks</u>

For a rectangular tanks with length (L) and breadth (B),

1) if the ratio of the length to breadth is more than 2 then the long wall is designed as cantilever and the short walls as slabs supported on long walls . Bottom portion of the short wall (H/4) or 1 m whichever is more is designed as cantilever.*(reinforced concrete structure I.C. Syal -A.K. Goel pa.535).*

2) For Tanks such as swimming pool , where L and B are both large , they should be designed independently as cantilever. *(advanced reinforced concrete design P. C. VARGHESE pa.487).*

Wall geometry

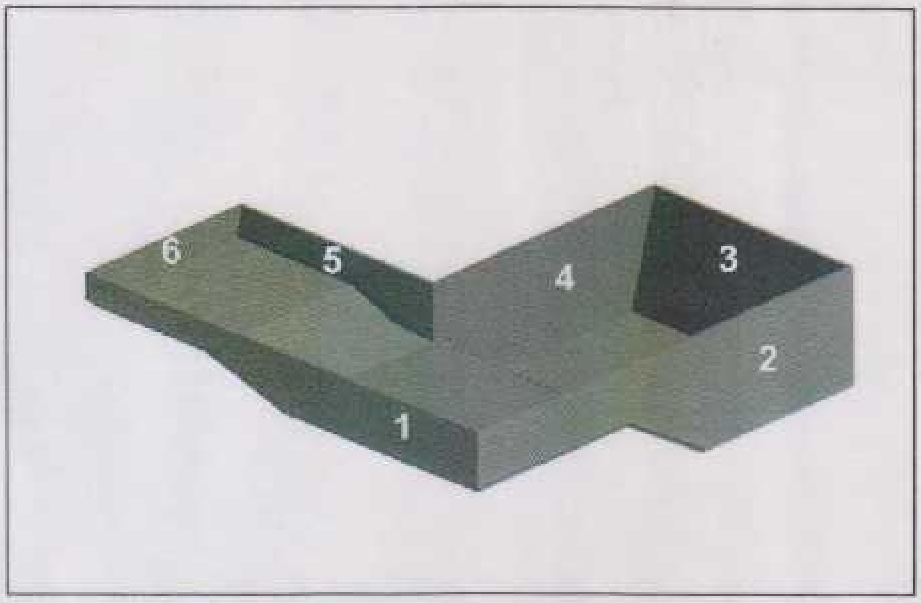
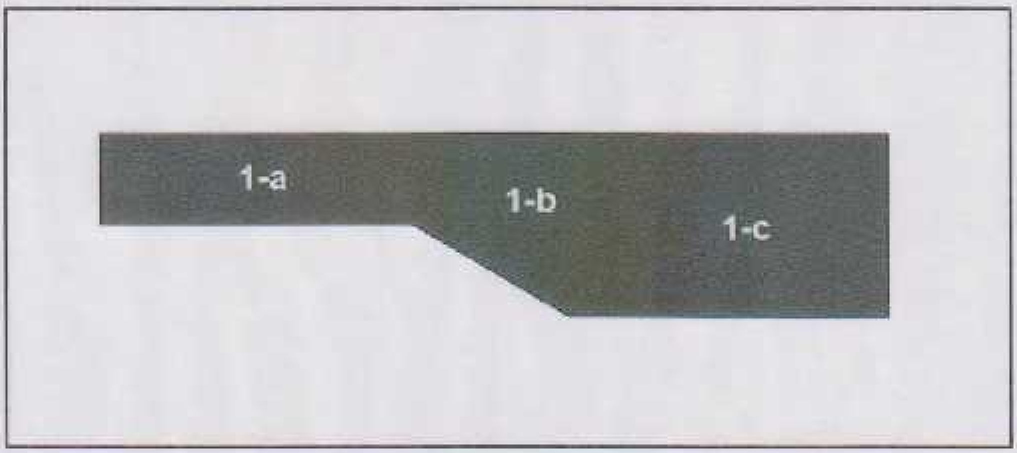
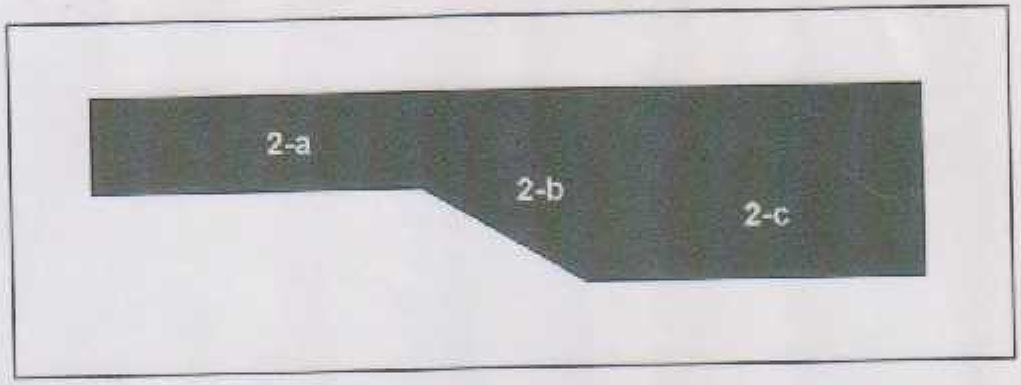


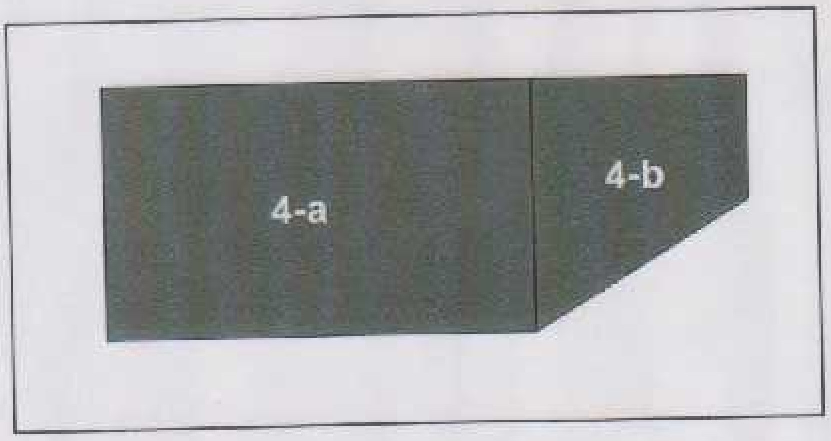
Figure (4-66): Wall geometry

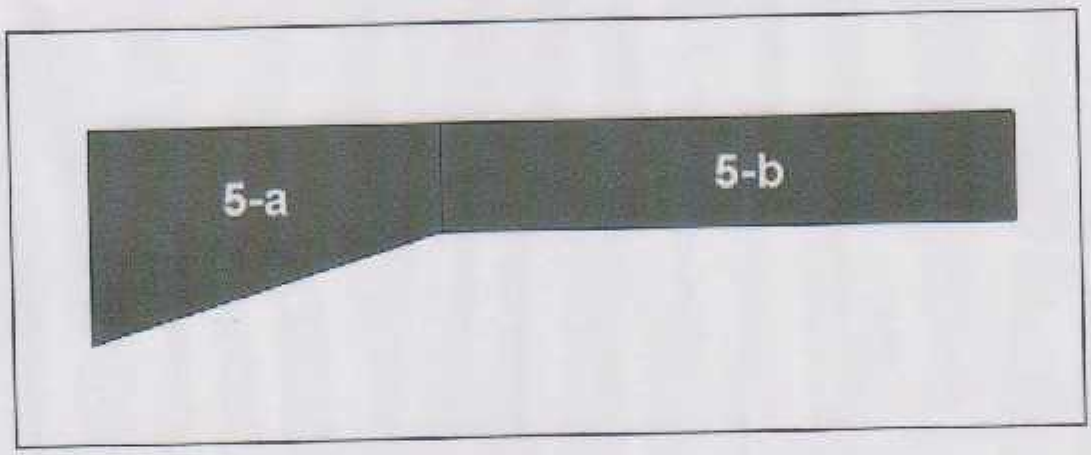
Depending upon the previous rules walls number 1,2,4 and 5 will be designed as cantilever wall while 3 and 6 will be designed as slab supported on the long walls.

The following figures show the walls segments due to its geometry variation:




2-a
2-b
2-c


4-a
4-b


5-a
5-b

Analysis Result:

Autodesk Robot Structural Analysis Professional is The software used to analyze the pool as whole . the program use the finite element method in finding the internal forces for the walls and base slab .the program consider the walls as a tow way slab so that we get internal forces in both vertical and horizontal directions . A screen capture for the model is shown.

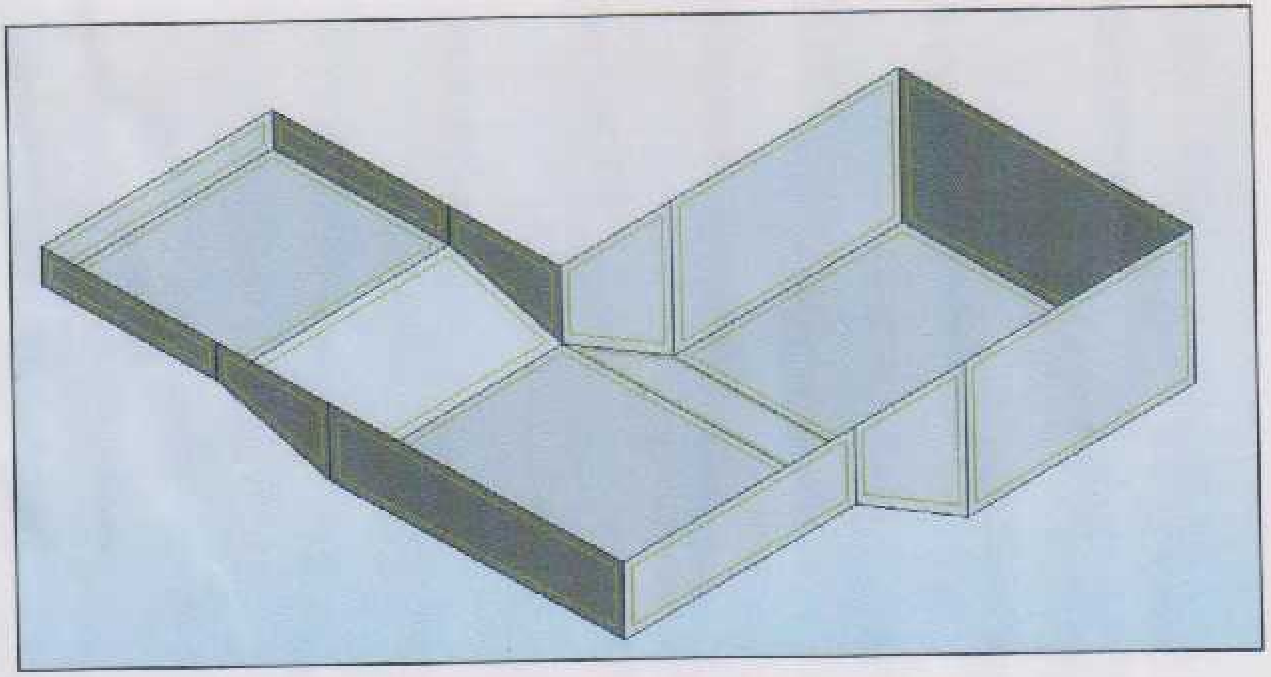


Figure (4-67): Analysis model of swimming pool

The mesh used in the model is 30 cm by 30cm to ensure the accuracy of the calculation.

1-) Case 1 results :

Deformed shape : the maximum deflection due to the service load in wall (2-c) is 1.4 cm which less the permissible deflection given in ACI-318 Table 9.5(b)

$$\frac{L}{180} = \frac{600}{180} = 3.3\ cm$$

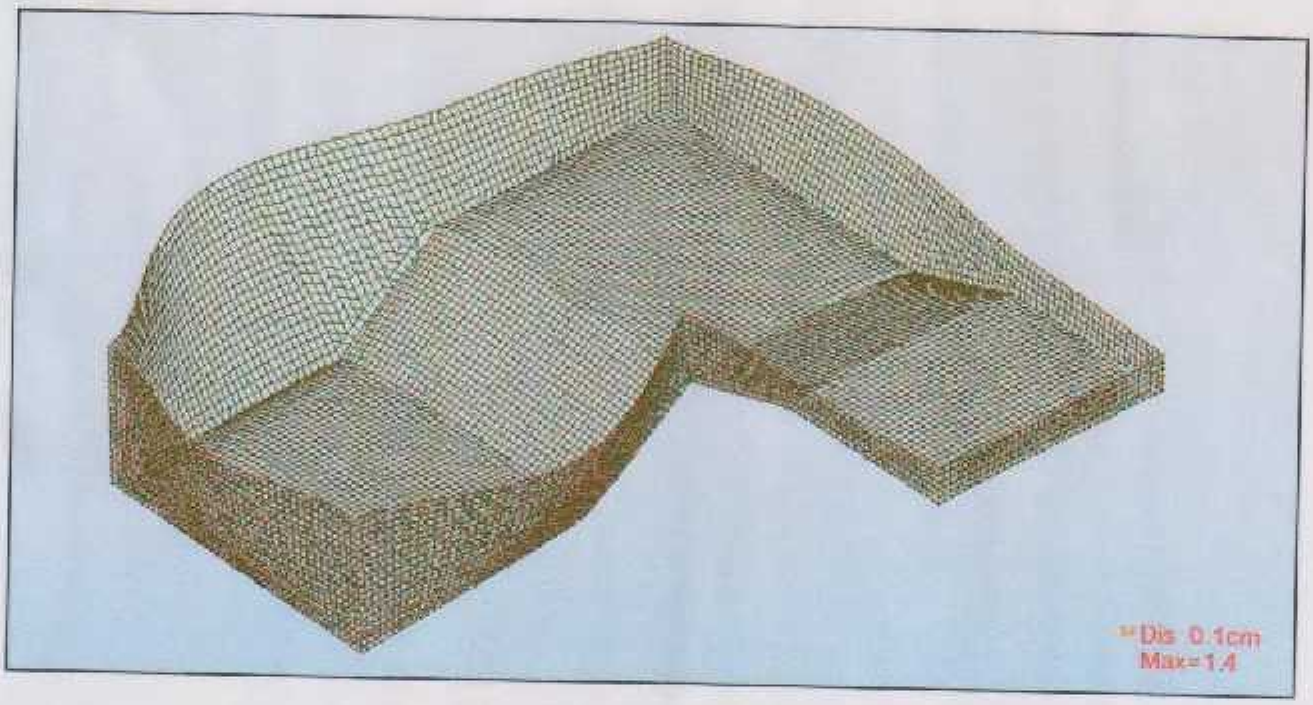


Figure (4-68): **Deformed shape due to case 1**

Soil Reaction: The maximum soil reaction on the base slabs due to service load is 157.6 (KN/m^2) is less than the allowable bearing capacity of the corresponding layer 186.4 (KN/m^2). (the base is displayed in an isolated view)

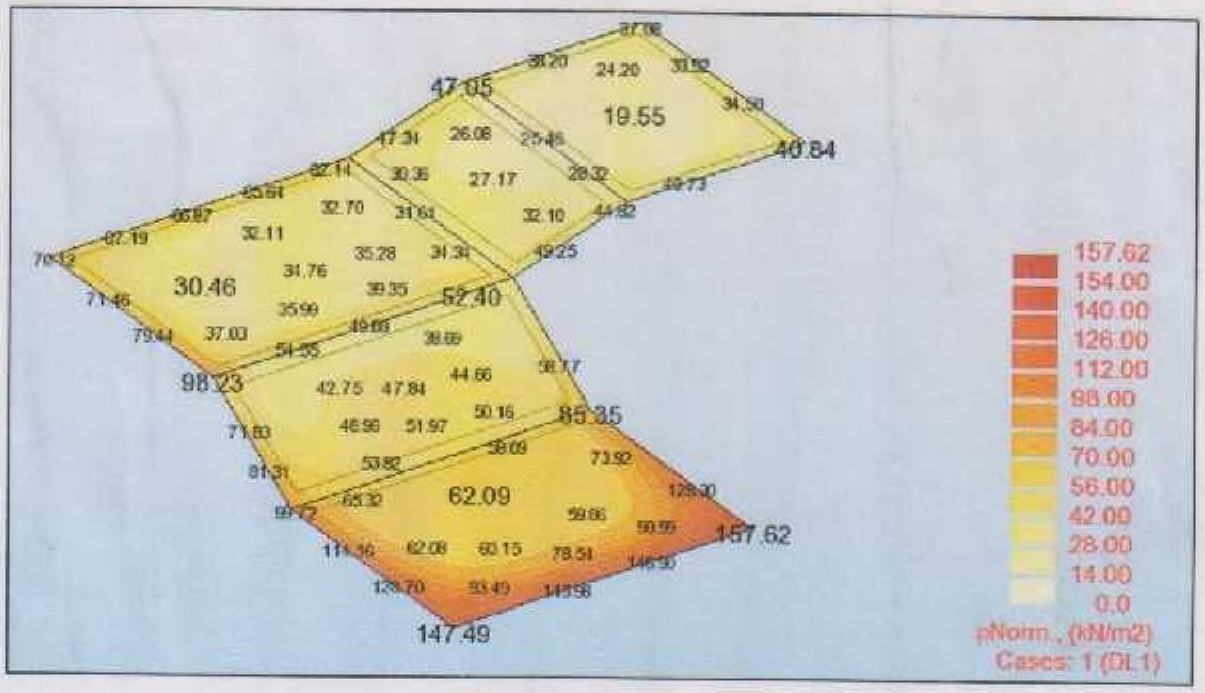


Figure (4-69): Soil reaction due to case 1

Walls Internal Forces: Wall 3 is showed in detailed as an example and the results for the other walls will be tabulated after in .the wall result is shown in an isolated view with a cut in the middle in vertical and horizontal direction .

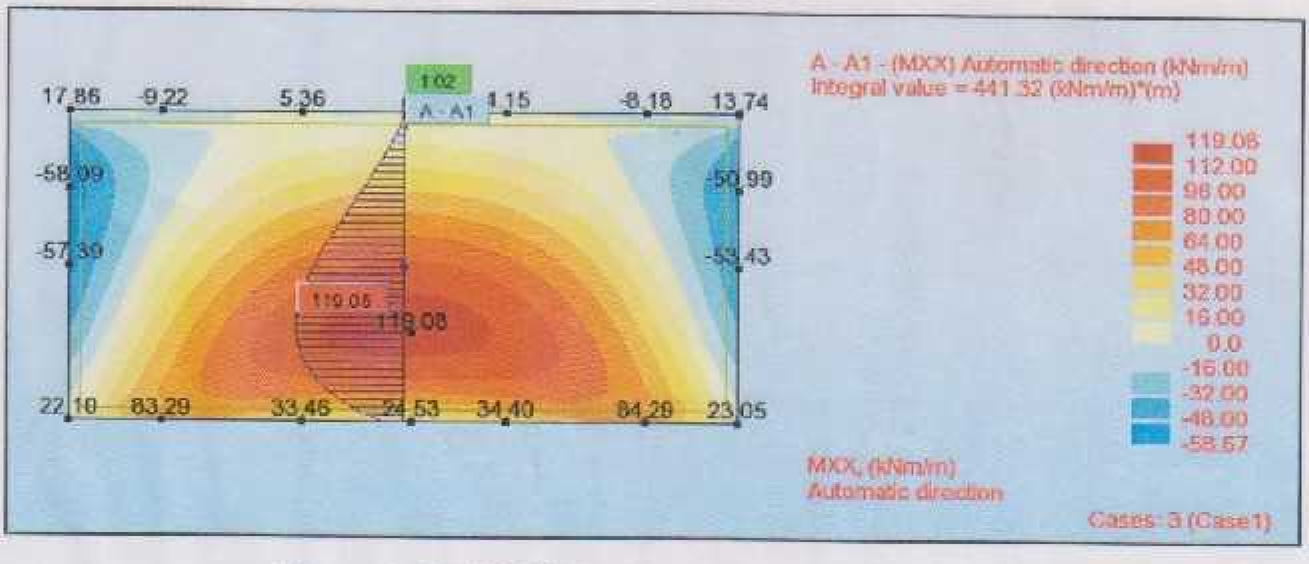


Figure (4-70): Wall 3 Bending moment MXX due to case 1

Mxx : which is used to determine the vertical reinforcement steel .

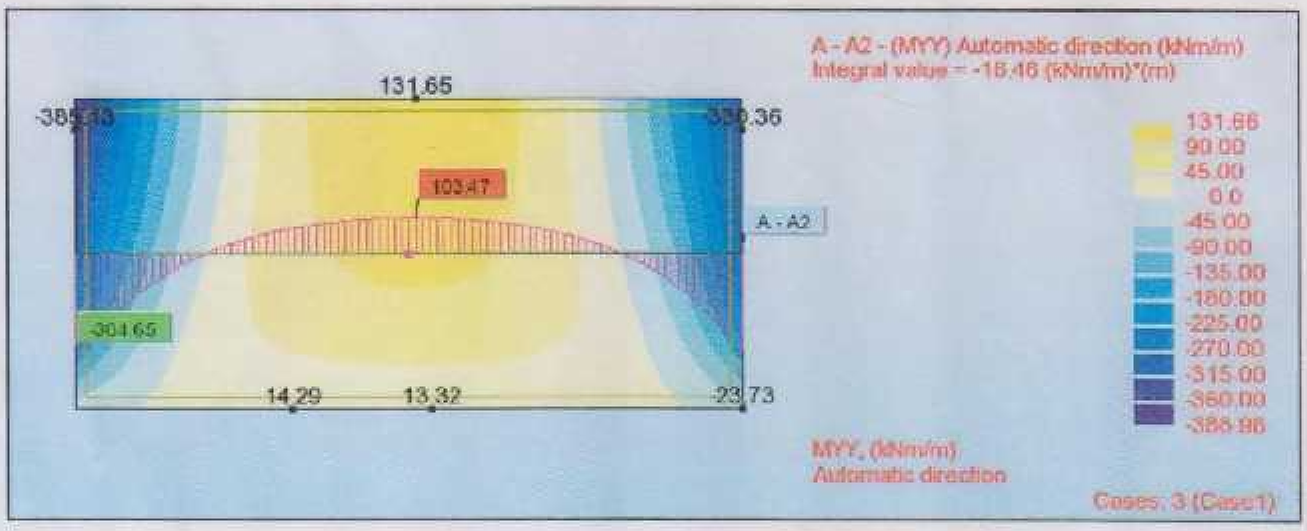


Figure (4-71): **Wall 3 Bending moment MYY due to case 1**

Myy : which is used to determine the horizontal reinforcement steel .

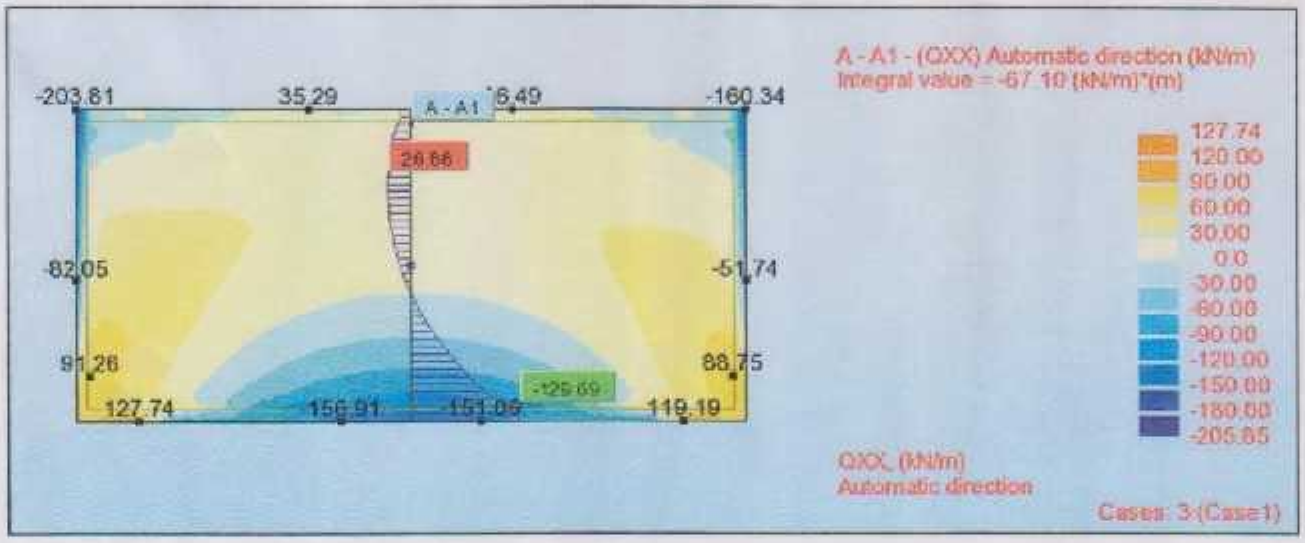


Figure (4-72): **Wall 3 Shear force QXX due to case 1**

Qxx :Shear force in the vertical direction .

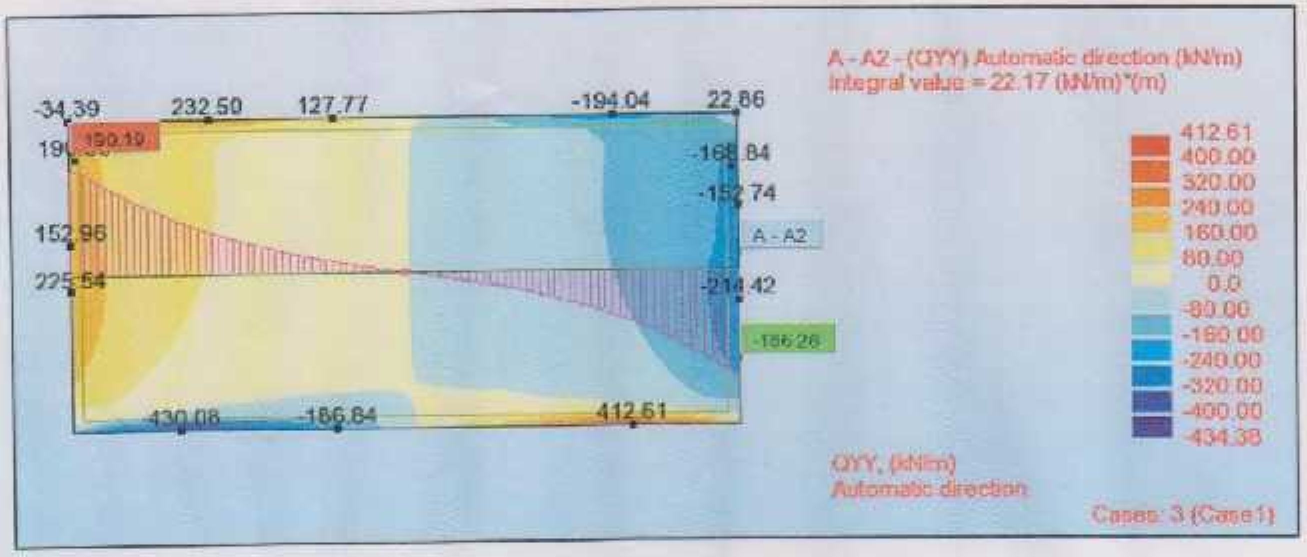


Figure (4-73): Wall 3 Shear force QYY due to case 1

Qyy :Shear force in the horizontal direction .

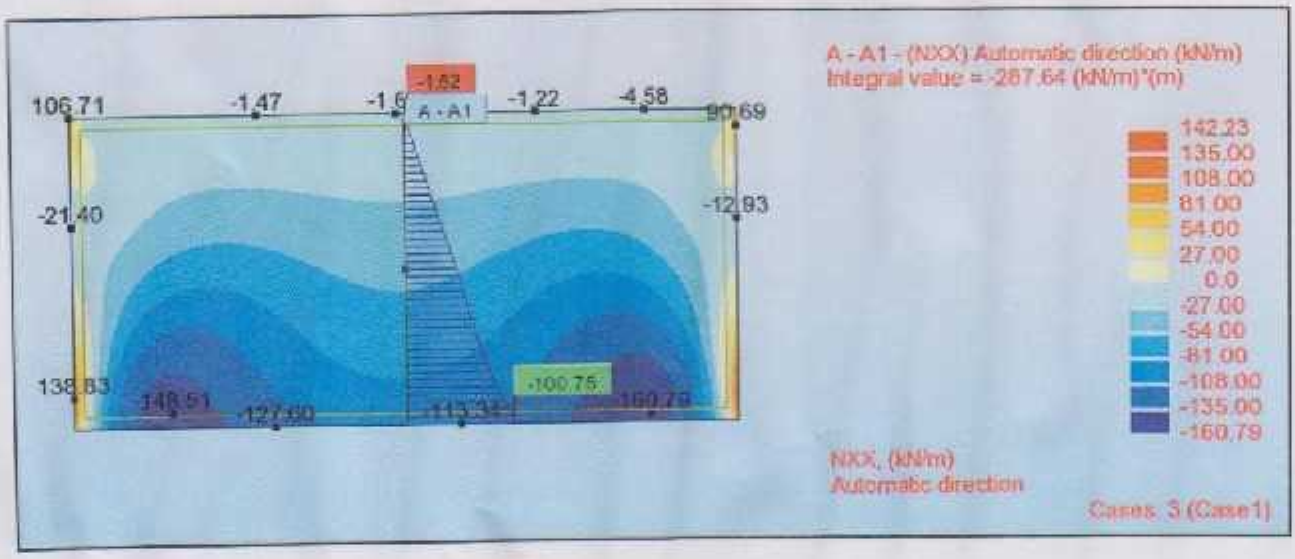


Figure (4-74): Wall 3 Tensile force NXX due to case 1

Nxx: tensile force in vertical direction

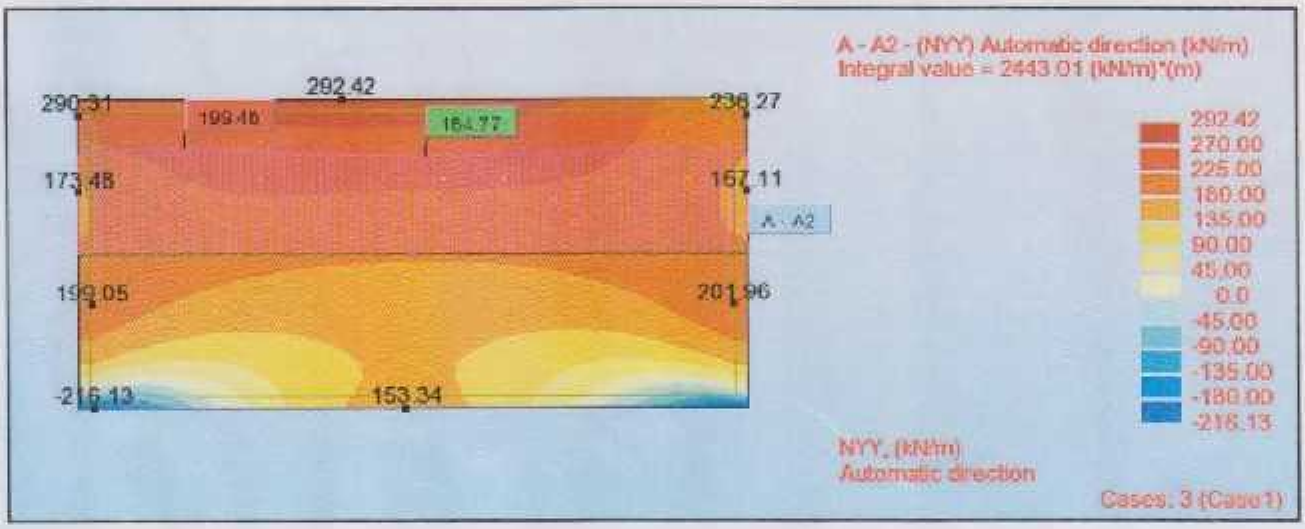


Figure (4-75): **Wall 3 Tensile force NYY due to case 1**

Nyy: tensile force in horizontal direction

Table (4-5): **Analysis result of walls due to case 1**

Wall name	Wall segment	Mxx*	Myy*	Qxx	Qyy	Nxx	Nyy
Wall 1	a	20	27.6	30.3	48.8	8.2	109.7
	b	32.4	18.57	34.3	42.6	35.9	173.9
	c	43.7	87.1	47.4	105	47.1	84
Wall 2	a	52	106.6	134.4	128	74	309
	b	131.9	113.04	120.1	136	238	560
	c	68	313	121.8	108	124	362
Wall 3	-	119.1	336.8	90.4	168.3	150.4	262.8
Wall 4	a	89.5	270.6	130	150.7	134.4	216.44
	b	105	130	209.2	232	180.95	431.9
Wall 5	a	66.6	104.6	166.7	81.5	139.2	418
	b	16.7	27.8	16.8	50.9	35.2	173
Wall 6	-	6	27.4	17.8	48.2	31.5	102.4

Sign of moment does not matter because the maximum value even positive or negative will be used in the design for both inside and outside layers.

It should be noted that the shear force (and so the corresponding tensile force) is taken at the critical section that is a distance "d" apart from the face of the support (base & adjacent walls).

2-) Case 2 results :

Deformed shape : the maximum deflection due to the service load in wall (2-c) is 0.8 cm which less the permissible deflection given in ACI-318 Table 9.5(b)

$$\frac{L}{180} = \frac{600}{180} = 3.3\ cm$$

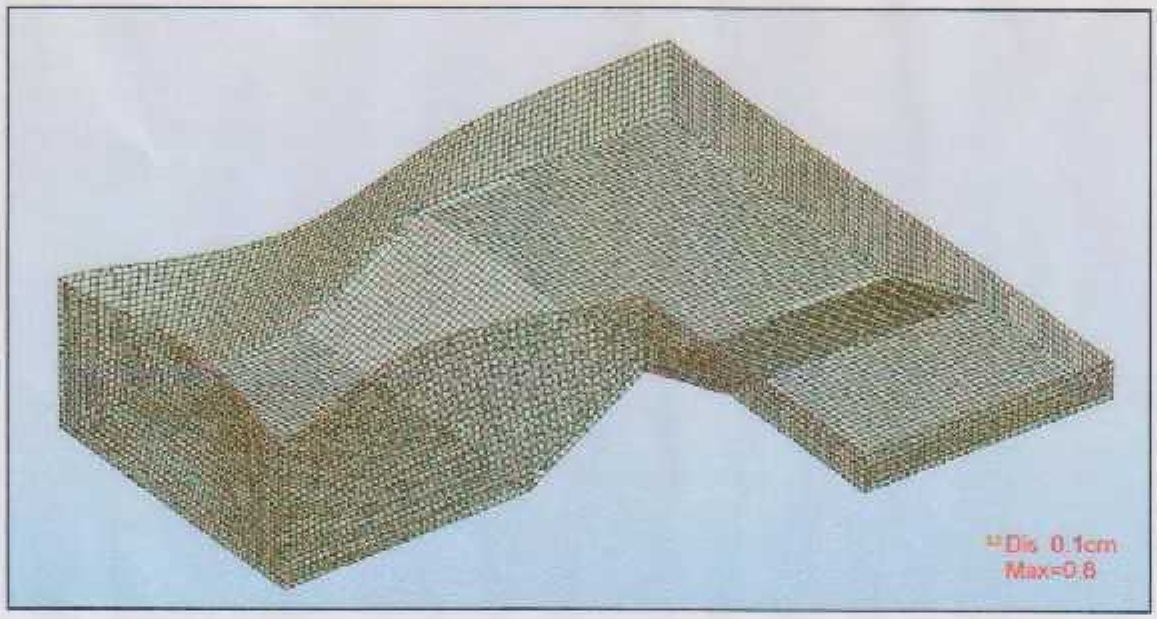


Figure (4-76): **Deformed shape due to case 2**

Soil Reaction : The maximum soil reaction on the base slabs due to service load is 38.83 (KN/m^2) is less than the allowable bearing capacity of the corresponding layer 186.4 (KN/m^2). (the base is displayed in an isolated view)

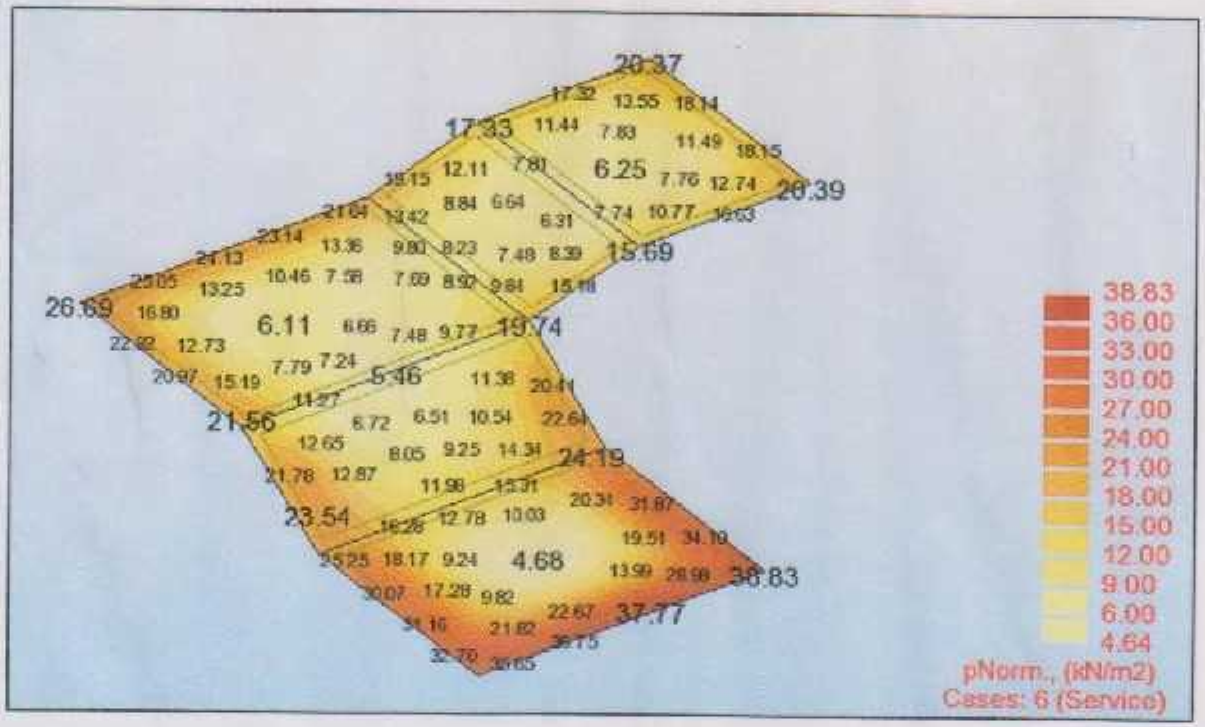


Figure (4-77): **Soil reaction due to case 2**

Walls Internal Forces : Wall 3 is showed in detailed as an example and the results for the other walls will be tabulated after in .the wall result is shown in an isolated view with a cut in the middle in vertical and horizontal direction .

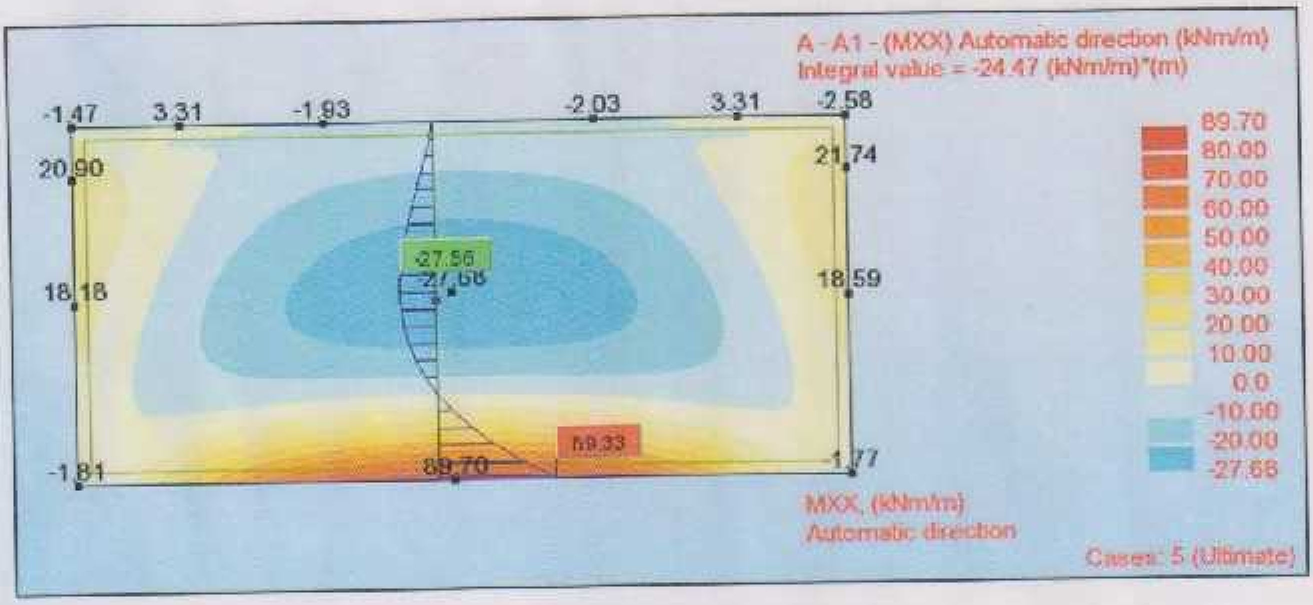


Figure (4-78): **Wall 3 Bending moment MXX due to case2**

Mxx : which is used to determine the vertical reinforcement steel .

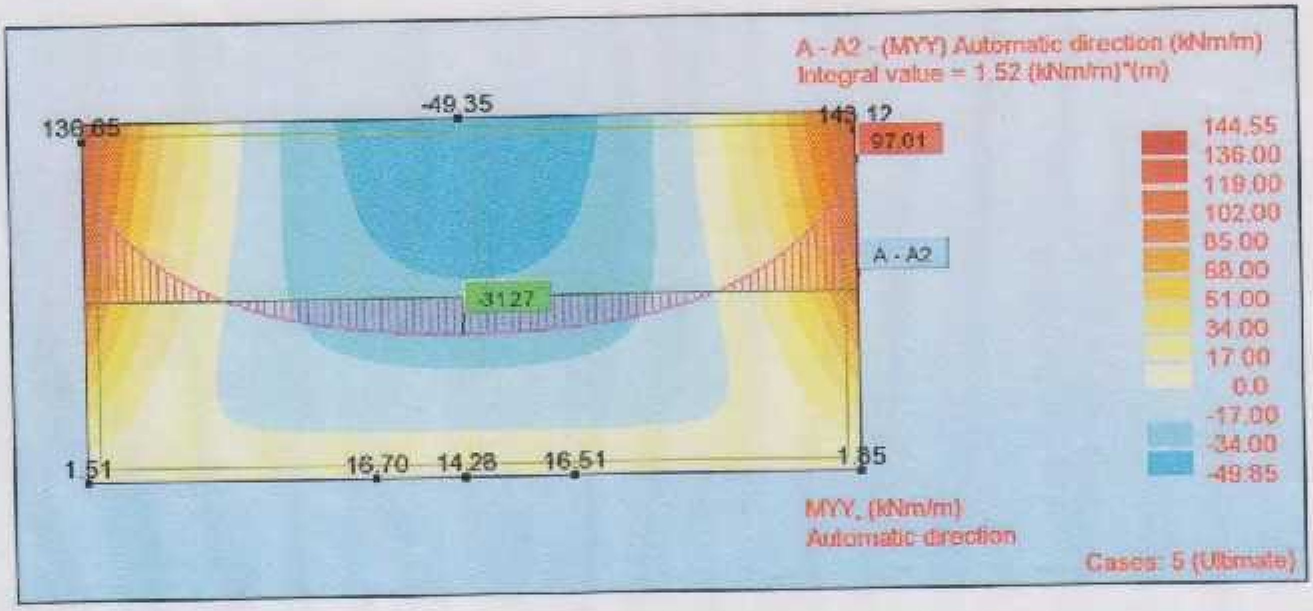


Figure (4-79): **Wall 3 Bending moment MYY due to case2**

Myy : which is used to determine the horizontal reinforcement steel

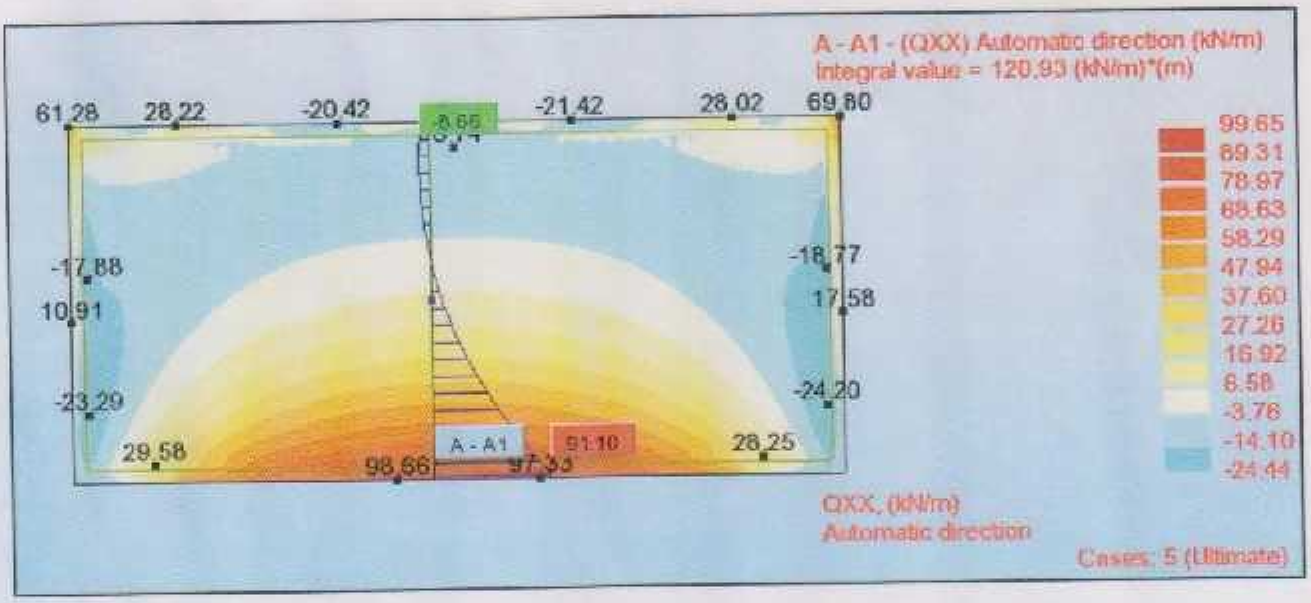


Figure (4-80): Wall 3 Shear force QXX due to case 2

Qxx :Shear force in the vertical direction .

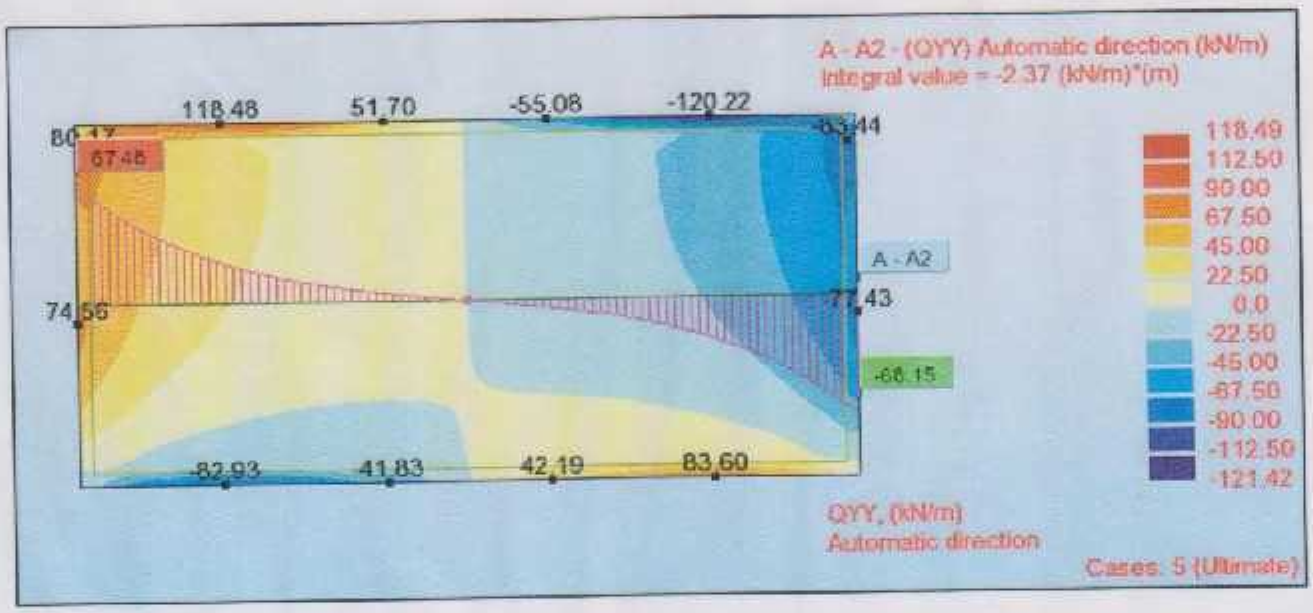


Figure (4-81): Wall 3 Shear force QYY due to case 2

Qyy :Shear force in the horizontal direction

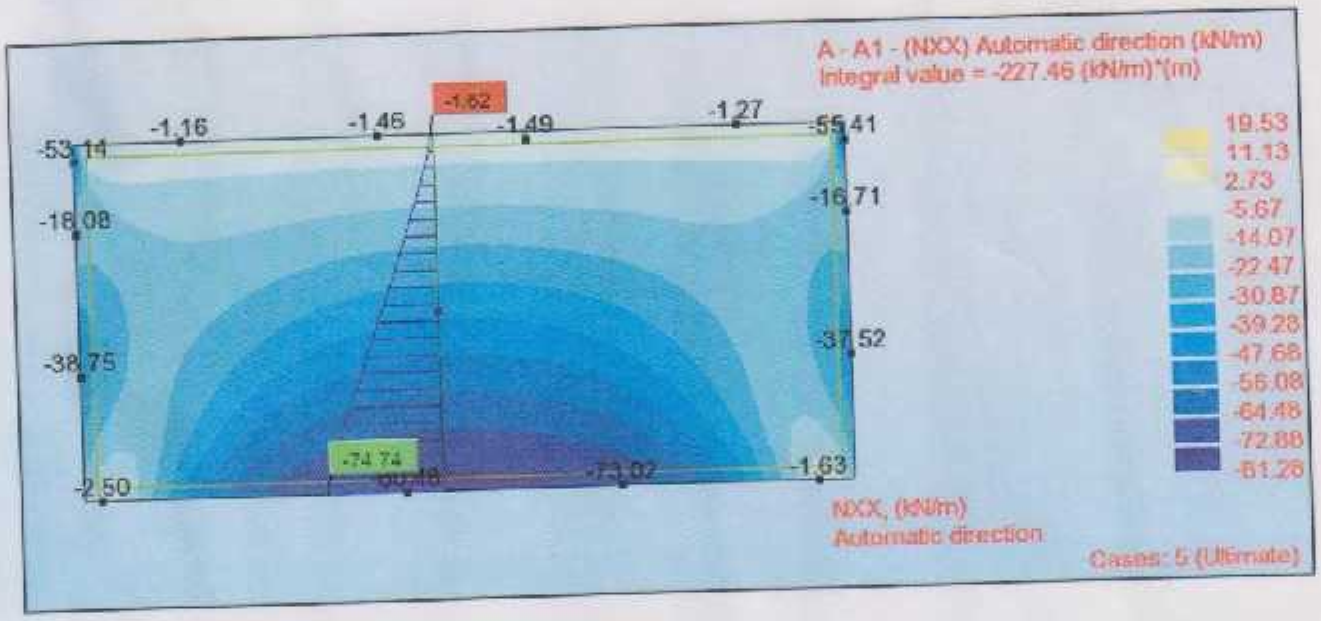


Figure (4-82): Wall 3 Tensile force NXX due to case 2

Nxx: tensile force in vertical direction

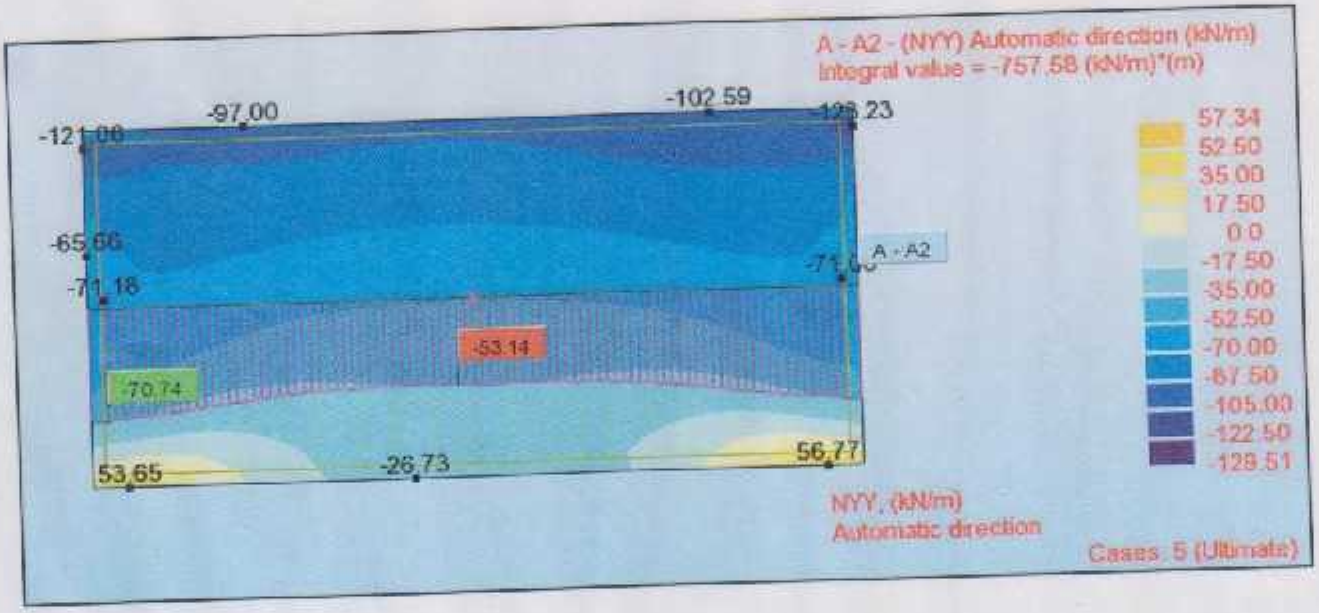


Figure (4-83): Wall 3 Tensile force NYY due to case 2

Nyy: tensile force in horizontal direction

Table (4-6): Analysis result of walls due to case 2

Wall name	Wall segment	Mxx	Myy	Qxx	Qyy	Nxx	Nyy
Wall 1	a	15	7	29.21	19.1	39.5	78.1
	b	18.5	6	24.1	21.7	32	55.1
	c	22	20.3	32	42.3	33.6	48.9
Wall 2	a	115.8	106	107	166.9	140.8	167
	b	100.1	42.2	75.4	136.2	83.1	182
	c	112.3	110.9	109.8	190.8	109.4	196
Wall 3	-	89.7	136.7	98.7	120.2	80.5	128.3
Wall 4	a	108.2	130	101.5	110	125.2	168
	b	77.75	94.7	105.7	121.2	130.9	201.3
Wall 5	a	38.7	84.4	108	65.9	201	163
	b	13	8.7	25	39.3	160.3	140.6
Wall 6	-	6	17.3	13.3	30.8	17.6	78.5

Table (4-7): Analysis result of walls due to Envelop case

Wall name	Wall segment	Mxx	Myy	Qxx	Qyy	Nxx	Nyy
Wall 1	a	20	27.6	30.3	48.8	39.5	109.7
	b	32.4	18.57	34.3	42.6	35.9	173.9
	c	43.7	87.1	47.4	105	47.1	84
Wall 2	a	115.8	106.6	134.4	166.9	140.8	309
	b	131.9	113.04	120.1	136.2	238	560
	c	112.3	313	121.8	190.8	124	362
Wall 3	-	119.1	336.8	98.7	168.3	150.4	262.8
Wall 4	a	108.2	270.6	130	150.7	134.4	216.44
	b	105	130	209.2	232	180.95	431.9
Wall 5	a	66.6	104.6	166.7	81.5	201	418
	b	16.7	27.8	25	50.9	160.3	173
Wall 6	-	6	27.4	17.8	48.2	31.5	102.4

4.3.5 Design of the pool walls

The design will be carried out according to the ultimate design method .the design steps will be shown in detailed for wall 3 as an example .

1- Design for Shear Force: it is designed so that no shear reinforcement is required.

a) shear force in vertical direction Qxx=98.7 KN/m and the corresponding tensile force Nxx=150.4 KN/m.

For 1 meter strip :-

Thickness t =50 cm , d=500-50-16 =434 mm.
Vc for member subjected to significant axial tension is given by 11.2.2.3 ACI-318-08

$$\emptyset \times Vc = 0.75 \times 0.17\left(1 + \frac{0.29 \times N_u}{A_g}\right) \times \sqrt{f_{c'}} \times b_w \times d$$

where Nu is negative for tension and Nu /Ag shall be expressed in MPa

$$\emptyset \times Vc = 0.75 \times 0.17\left(1 + \frac{0.29 \times -150400}{500 \times 1000}\right) \times \sqrt{24} \times 1000 \times 434$$

$$\emptyset \times Vc = 247.44 \text{ KN} > Qxx = 98.7 \rightarrow \text{thickness} = 50 \text{ cm is safe.}$$

b) shear force in horizontal direction Qyy=168.3 KN/m and the corresponding tensile force Nyy=262.8 KN/m .

$$\emptyset \times Vc = 0.75 \times 0.17\left(1 + \frac{0.29 \times -262800}{500 \times 1000}\right) \times \sqrt{24} \times 1000 \times 434$$

$$\emptyset \times Vc = 229.8\ KN > Qyy = 168.3 \rightarrow thickness\ is\ safe.$$

2- Design for Vertical Bending Moment (Vertical Steel) :

Mxx=119.1 KN.m/m

$$Kn = \frac{\frac{M_u}{\emptyset}}{bd^2} = \frac{119.1 \times \frac{10^6}{0.9}}{1000 \times (434)^2} = 0.7 \text{ Mpa}$$

$$m = \frac{f_y}{0.85f_c'} = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho_{required} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 0.7 \times 20.6}{420}}\right)$$
$$= 0.0017$$

$$A_{s\,required} = \rho_{required} \times b \times d = 0.0017 \times 100 \times 43.4 = 7.4\ \frac{cm^2}{m}$$

$$A_{s\,minimum} = 0.0012 \times 100 \times 50 = 6\ \frac{cm^2}{m}\ , \qquad eq. 14.3.2\ ACI\ 318-08$$

Select $\emptyset 14\ mm$ with $As = \frac{1.4^2 \times \pi}{4} = 1.54\ cm^2$

$$n = \frac{7.4}{1.54} = 4.8 \approx 5\emptyset 14 \text{ bars for 1 meter width}$$

$\therefore$ use $\emptyset 14@20$cm for both inside and outside layers

check maximum spacing :

$$s = 380\left(\frac{280}{f_s}\right) - 2.5c_c = s = 380\left(\frac{280}{280}\right) - 2.5 \times 50 = 25.5\ \text{cm} > 20cm \rightarrow O.K$$

3.Design for Horizontal Bending Moment (horizontal steel):

Myy=336.8 KN.m/m

$$\mathrm{Kn} = \frac{\frac{M_u}{\emptyset}}{bd^2} = \frac{336.8 \times \frac{10^6}{0.9}}{1000 \times (434)^2} = 1.99 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho_{required} = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 1.99 \times 20.6}{420}}\right) = 0.005$$

$$A_{s\,required} = \rho_{required} \times b \times d = 0.0049 \times 100 \times 43.4 = 21.64 \ \text{cm}^2/m$$

$$A_{s\,minimum} = 0.001 \times 100 \times 50 = 5\ \frac{\text{cm}^2}{m}, \qquad eq. 14.3.3\ ACI\ 318 - 08$$

Select Ø16 *mm* with As = 2.01 cm^2

$$n = \frac{21.64}{2.01} = 11\emptyset 16 \text{ bars for 1 meter length}$$

$\therefore$ *use* Ø16@9*cm* for both inside and outside layers

For the other walls an excel sheet is made in order to simplify the calculation .

Table (4-8): Walls Reinforcement Results

Wall name	Wall segment	Thickness cm	Vertical steel	Horizontal steel
Wall 1	a	30	Ø10@20cm	Ø10@20cm
	b	30	Ø10@20cm	Ø10@20cm
	c	30	Ø10@15cm	Ø16@15cm
Wall 2	a	50	Ø12@15cm	Ø12@15cm
	b	50	Ø14@15cm	Ø14@20cm
	c	50	Ø14@20cm	Ø16@10cm
Wall 3	-	50	Ø14@20cm	Ø16@9cm
Wall 4	a	50	Ø12@15cm	Ø16@10cm
	b	50	Ø12@15cm	Ø16@15cm
Wall 5	a	30	Ø12@15cm	Ø12@15cm
	b	30	Ø12@15cm	Ø12@20cm
Wall 6	-	30	Ø12@15cm	Ø12@20cm

4.3.6 Structural Analysis of the base slabs

Analysis result:

Base Slabs Internal Forces: Slab at level "–6.0 m "is showed in detailed as an example and the results for the other base slabs will be tabulated after in .the slab result is shown in an isolated view with a cut in the middle in longitudinal and lateral directions .

1-) Case 1 results :

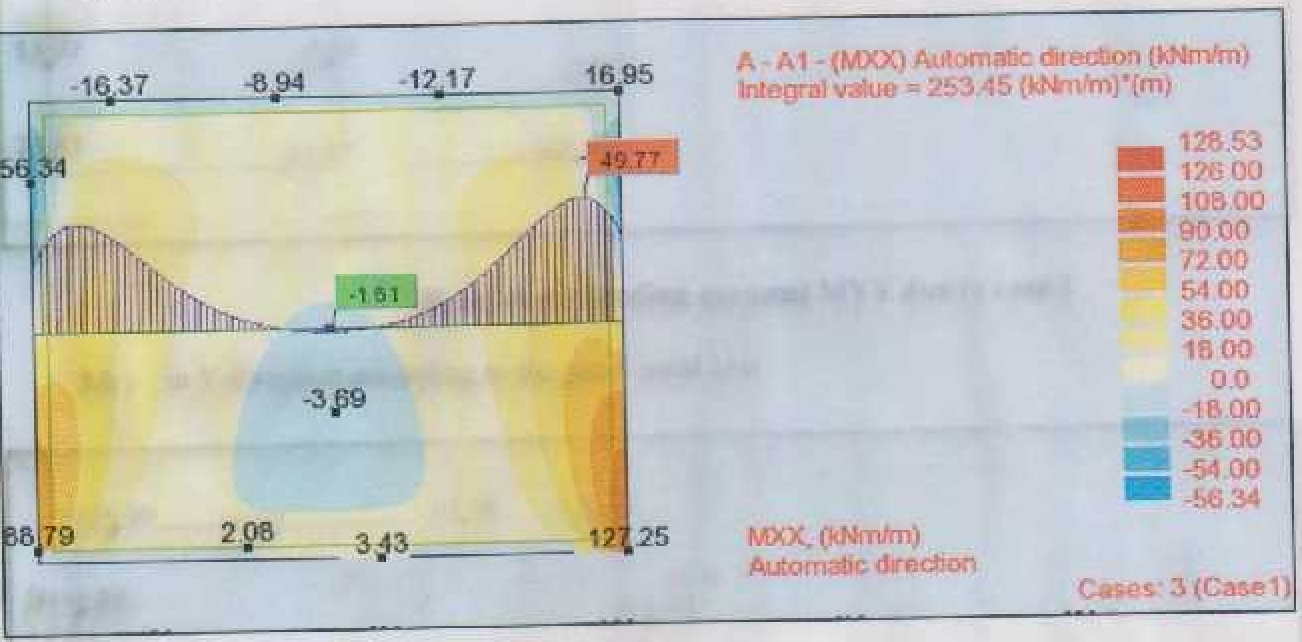
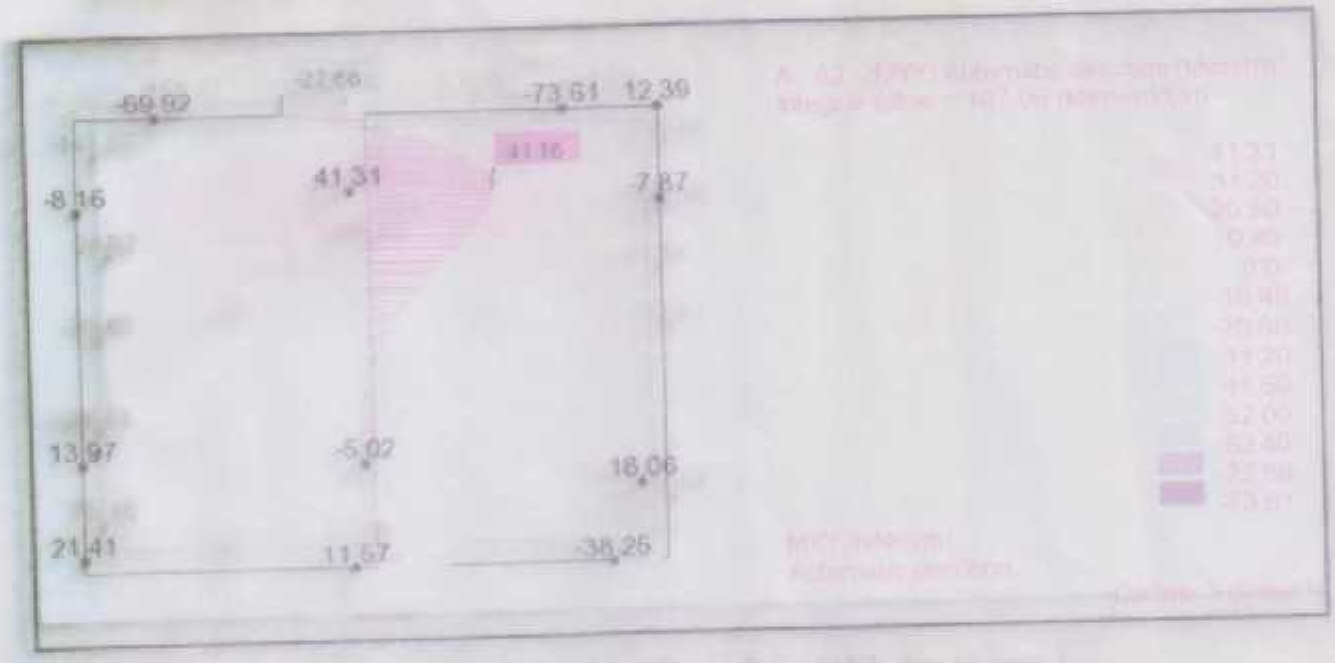


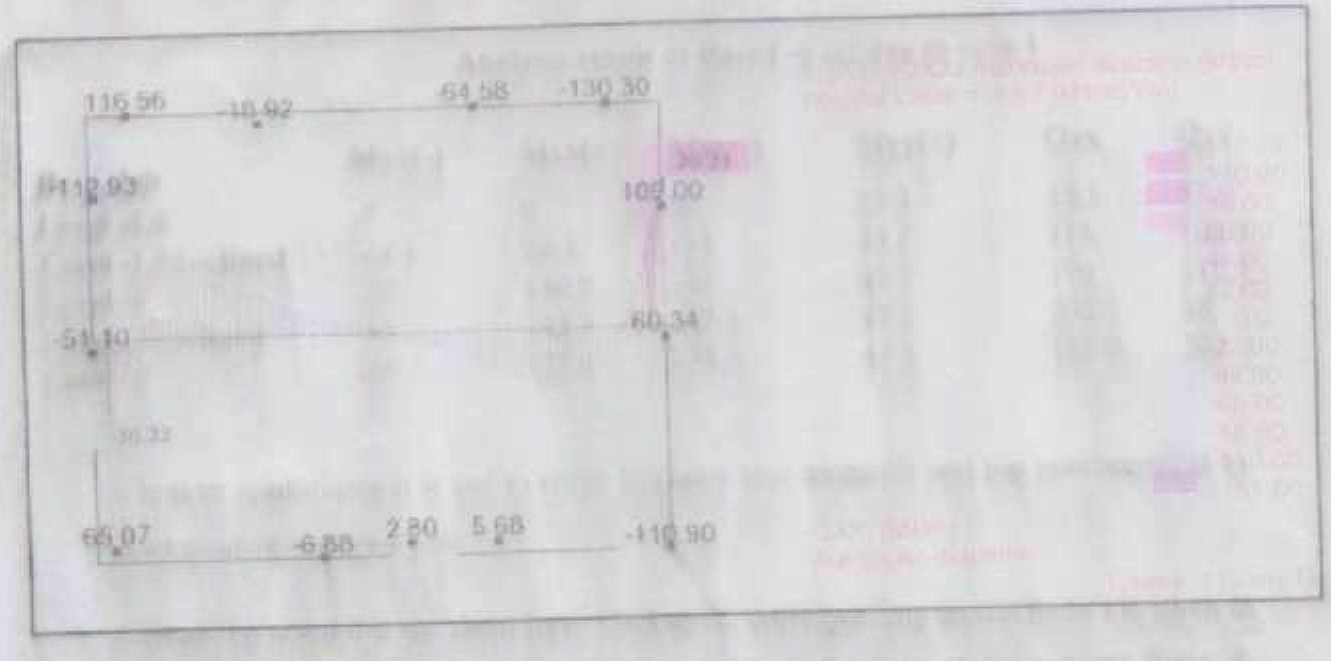
Figure (4-84): **Base(-6 m) bending moment MXX due to case 1**

Mxx : in X-direction according to the panel local axis .



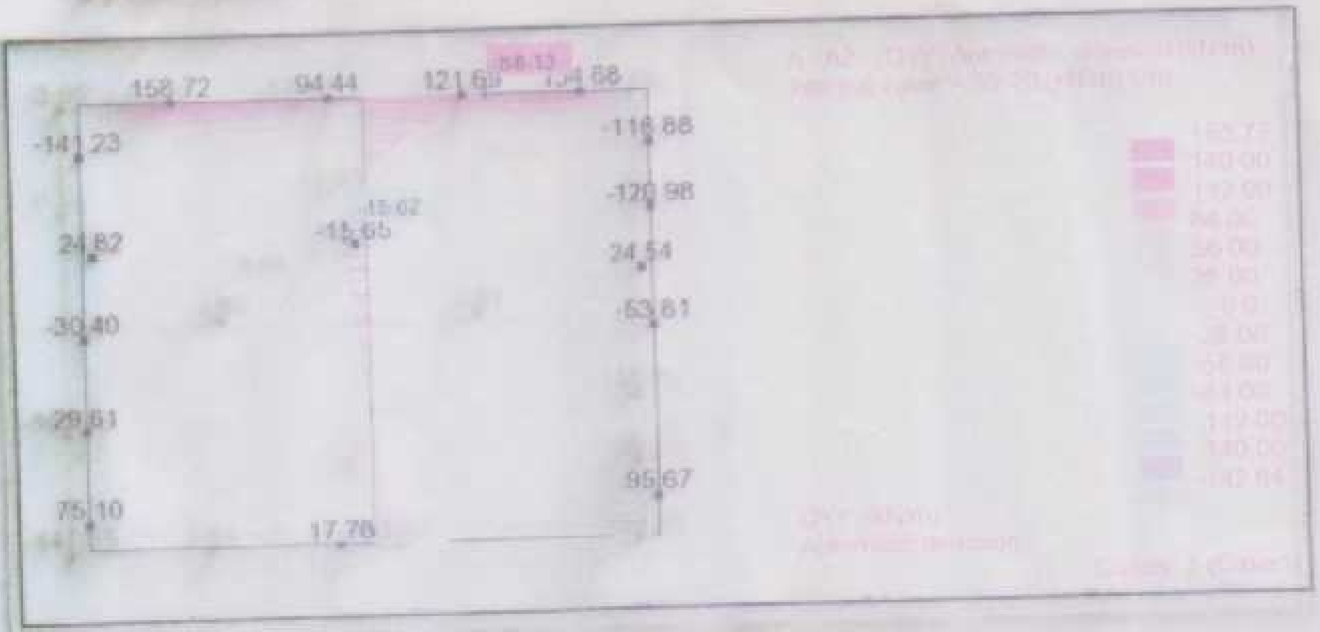
Base(-6 m) bending moment MYY due to case 1

Myy : in Y-direction according to the panel local axis .



Base(-6 m) Shear force QXX due to case 1

Qxx : in X-direction according to the panel local axis



Base(-6 m) Shear force QYY due to case 1

Qyy : in Y-direction according to the panel local axis

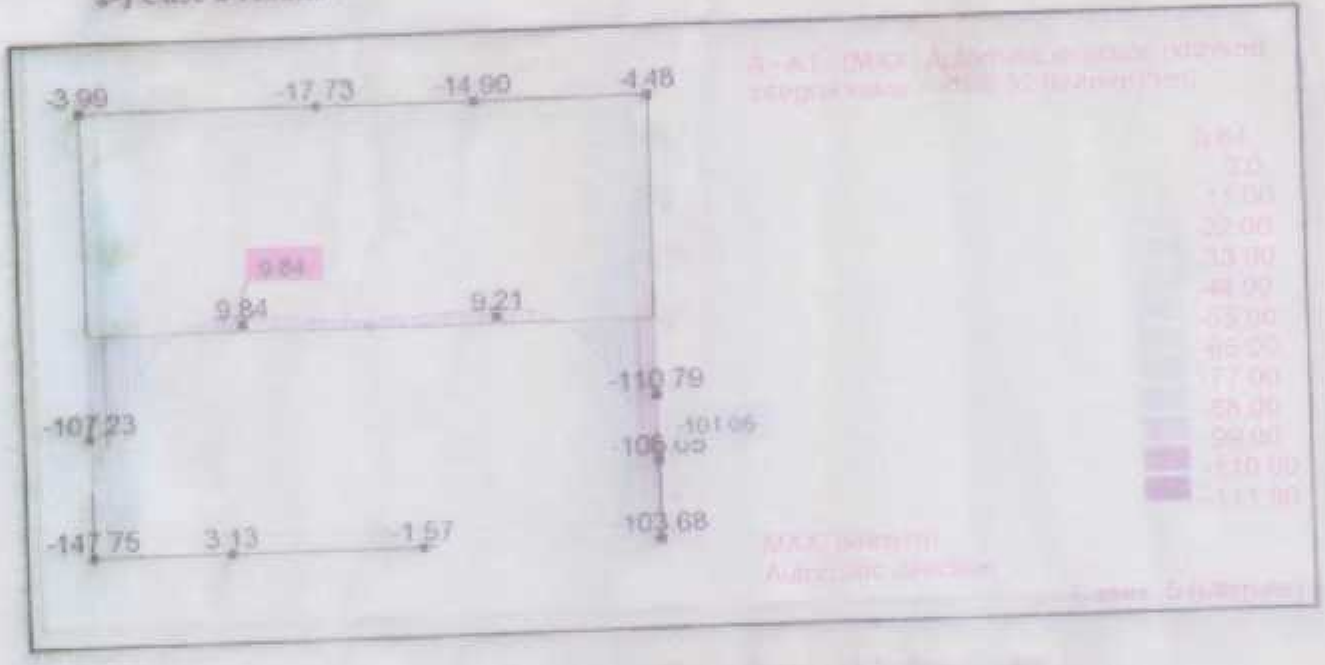
Analysis result of Base (-6 m) due to case 1

Base slab	Mxx(-)	Mxx(+)	Myy(-)	Myy(+)	Qxx	Qyy
Level -1.5	-7	9	-6	23.1	42.1	37.6
Level -1.5/inclined	-44.1	20.1	-13	43.5	115	160.8
Level -3	-20	140.9	-42	43.7	170	110.4
Level -3/inclined	-55	148.3	-57.3	42.2	205.3	58
Level -6	-56	127.4	-73.3	41.3	128.9	202.1

A bottom reinforcement is put to resist Negative sign moments and top reinforcement to resist positive sign moments .

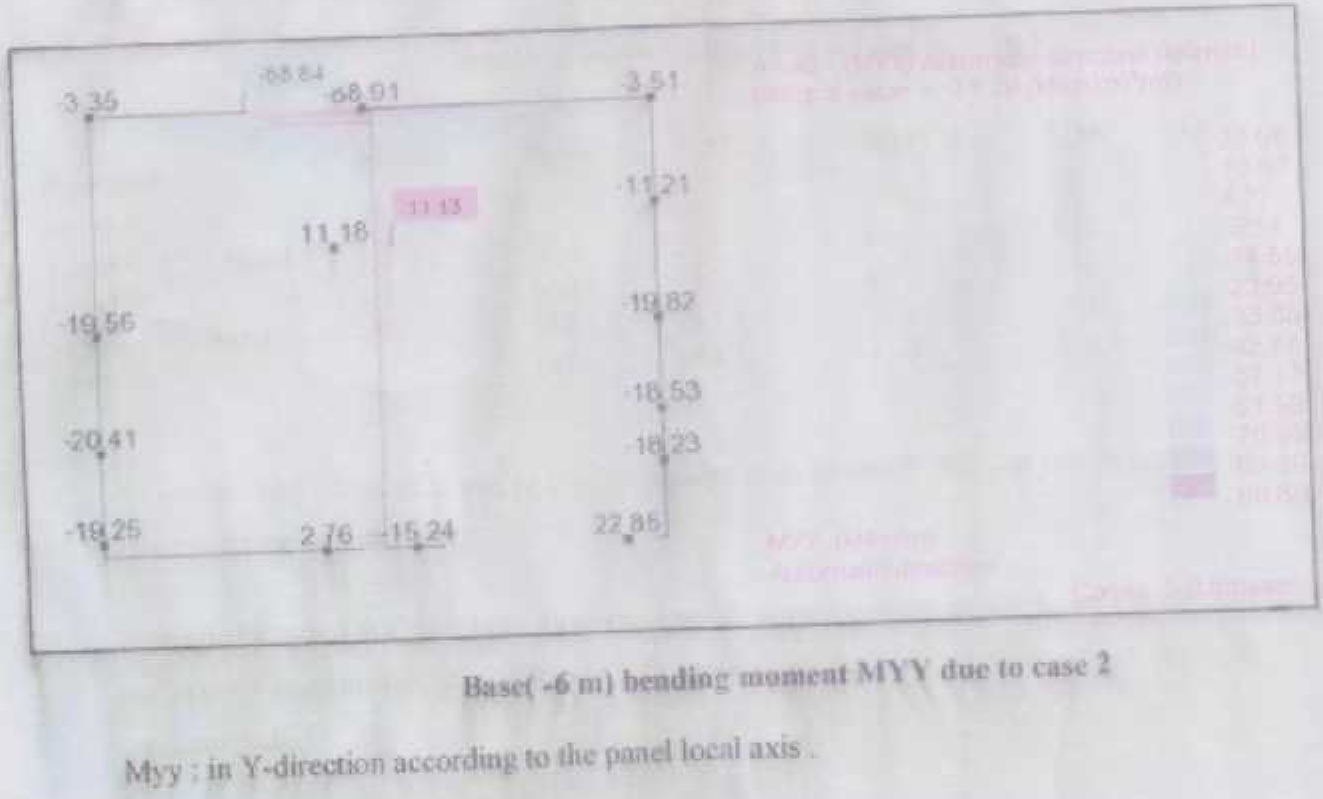
It should be noted that the shear force (and so the corresponding tensile force) is taken at the critical section that is a distance "d" apart from the face of the support (base & adjacent walls).

2-) Case 2 results :



Base(-6 m) bending moment MXX due to case 2

Mxx : in X-direction according to the panel local axis .



Base(-6 m) bending moment MYY due to case 2

Myy : in Y-direction according to the panel local axis .

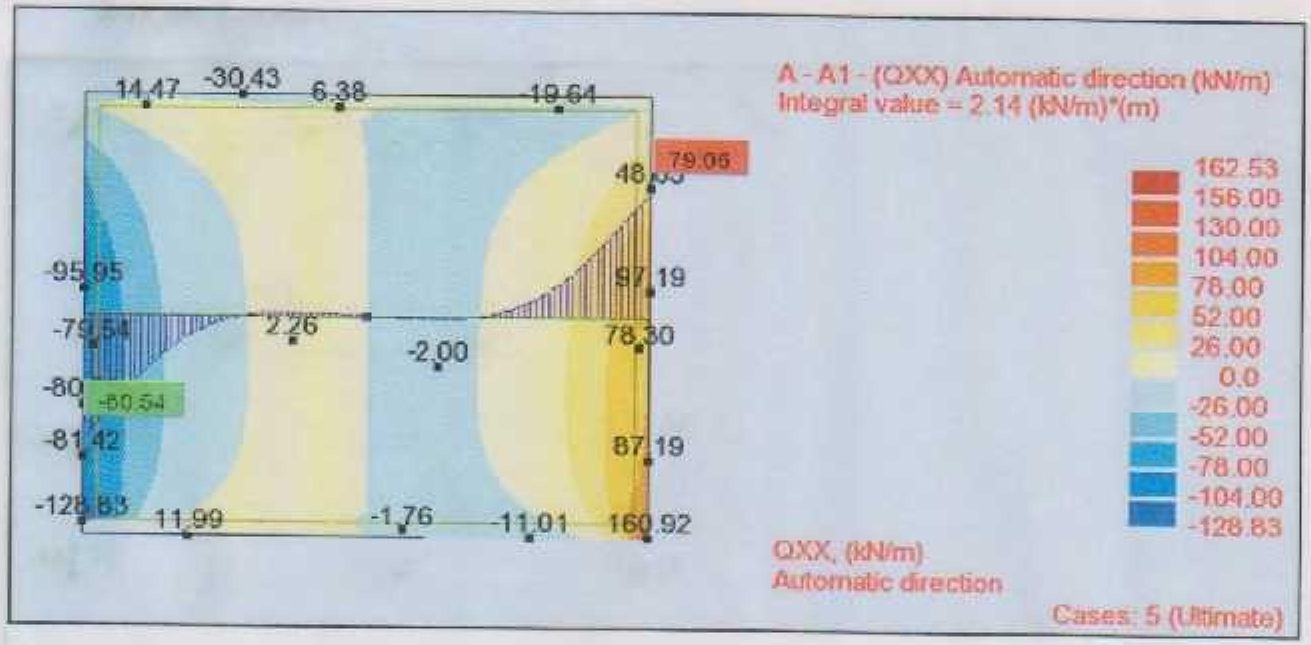


Figure (4-90): Base(-6 m) Shear force QXX due to case 2

Qxx : in X-direction according to the panel local axis .

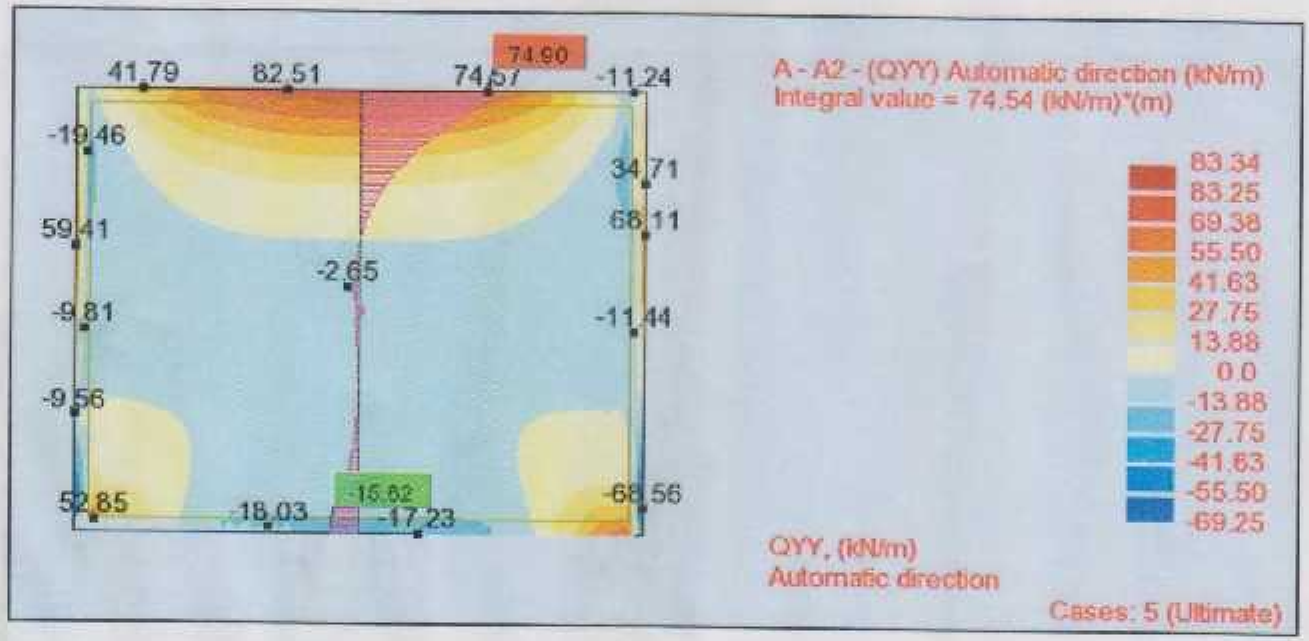


Figure (4-91): Base(-6 m) Shear force QYY due to case 2

Qyy : in Y-direction according to the panel local axis .

Table (4-10): Analysis result of Base (-6 m) due to case 2

Base slab	Mxx(-)	Mxx(+)	Myy(-)	Myy(+)	Qxx	Qyy
Level -1.5	-13.3	3.7	-17.9	3.4	17.7	30.7
Level -1.5/inclined	-15.2	3.12	-40.2	44.63	30	31.8
Level -3	-142.9	4.98	-30.6	23	130	129.5
Level -3/inclined	-139.3	38.3	-46.6	24.3	195	123.3
Level -6	-143.4	9.8	-88.9	22.8	203	102.4

Table (4-11): Analysis result of Base (-6 m) due to Envelop case

Base slab	Mxx(-)	Mxx(+)	Myy(-)	Myy(+)	Qxx	Qyy
Level -1.5	-13.3	9	-17.9	23.1	42.1	37.6
Level -1.5/inclined	-44.1	20.1	-40.2	44.63	115	160.8
Level -3	-142.9	140.9	-42	43.7	170	129.5
Level -3/inclined	-139.3	148.3	-57.3	42.2	205.3	123.3
Level -6	-143.4	127.4	-88.9	41.3	203	202.1

4.3.7 Design of the pool base slabs

The design will be carried out according to the ultimate design method .the design steps will be shown in detailed for Slab at "level -6" m as an example .

1- Design for Shear Force: it is designed so that no shear reinforcement is required.

a) shear force in X- direction Qxx = 203 KN/m

For 1 meter strip :-

thickness t =30 cm , d=300-50-16 =234mm.

Vc for member subjected to significant axial tension is given by (11.2.1.1 ACI-318-08)

$$\emptyset \times Vc = 0.75 \times 0.17 \times \sqrt{f_{c'}} \times b_w \times d$$

$$\emptyset \times Vc = 0.75 \times 0.17 \times \sqrt{24} \times 1000 \times 234$$

$$\emptyset \times Vc = 146.2\ KN < Qxx = 203 \rightarrow \text{thickness must be increased.}$$

Take h=40 cm : d=400-50-16 =334mm

$$\emptyset \times Vc = 0.75 \times 0.17 \times \sqrt{24} \times 1000 \times 334$$

$$\emptyset \times Vc = 204.5\ KN > Qxx = 203 \rightarrow thickness\ is\ safe.$$

b) shear force in Y- direction Qyy=202.1 KN/m

$$\emptyset \times Vc = 0.75 \times 0.17 \times \sqrt{24} \times 1000 \times 334$$

$$\emptyset \times Vc = 204.5\ KN > Qyy = 202.1 \rightarrow thickness\ is\ safe$$

2- Design for X-Direction negative Bending Moment (Bottom reinforcement) :

Mxx = - 143.4 KN.m/m

$$\text{Kn} = \frac{\frac{M_u}{\emptyset}}{bd^2} = \frac{143.4 \times \frac{10^6}{0.9}}{1000 \times (334)^2} = 1.43 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho_{required} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 1.43 \times 20.6}{420}}\right) = 0.0035$$

$$A_{s\ required} = \rho_{required} \times b \times d = 0.0035 \times 100 \times 33.4 = 11.8\ cm^2/m$$

$$A_{s\ minimum} = 0.0018 \times 100 \times 40 = 7.2\ \frac{cm^2}{m}, \quad eq - 7.12.2.1\ ACI\ 318 - 08$$

Select $\emptyset 16\ mm$ with $As = \frac{1.6^2 \times \pi}{4} = 2\ cm^2$

$n = \frac{11.8}{2} = 5.9 \approx 6\emptyset 16$ bars for 1 meter width

$\therefore use\ \emptyset 16@15\ cm$

check maximum spacing :

$$s = 380\left(\frac{280}{f_s}\right) - 2.5c_c = s = 380\left(\frac{280}{280}\right) - 2.5 \times 50 = 25.5\ \text{cm} > 15cm \rightarrow O.K$$

3- Design for X-Direction Positive Bending Moment (Top reinforcement) :

Mxx = 127.4 KN.m/m

$$\text{Kn} = \frac{\frac{M_u}{\emptyset}}{bd^2} = \frac{127.4 \times \frac{10^6}{0.9}}{1000 \times (334)^2} = 1.27\ \text{Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho_{required} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 1.27 \times 20.6}{420}}\right) = 0.0031$$

$$A_{s\ required} = \rho_{required} \times b \times d = 0.0031 \times 100 \times 33.4 = 10.43\ \text{cm}^2/m$$

$$A_{s\ minimum} = 0.0018 \times 100 \times 40 = 7.2\ \frac{\text{cm}^2}{m}, \qquad eq - 7.12.2.1\ ACI\ 318 - 08$$

Select $\emptyset 16\ mm$ with $As = \frac{1.6^2 \times \pi}{4} = 2\ cm^2$

$n = \frac{10.43}{2} = 5.2 \approx 6\emptyset 16$ bars for 1 meter width

$\therefore use\ \emptyset 16@15\ cm$

check maximum spacing :

$$s = 380\left(\frac{280}{f_s}\right) - 2.5c_c = s = 380\left(\frac{280}{280}\right) - 2.5 \times 50 = 25.5\ \text{cm} > 15cm \rightarrow O.K$$

4- Design for Y-Direction negative Bending Moment (Bottom reinforcement) :

Mxx=- 88.9 KN.m/m

$$\text{Kn} = \frac{\frac{M_u}{\emptyset}}{bd^2} = \frac{88.9 \times \frac{10^6}{0.9}}{1000 \times (334)^2} = 0.88 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho_{required} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 0.88 \times 20.6}{420}}\right) = 0.0021$$

$$A_{s\,required} = \rho_{required} \times b \times d = 0.0021 \times 100 \times 33.4 = 7.16 \ \text{cm}^2/m$$

$$A_{s\,minimum} = 0.0018 \times 100 \times 40 = 7.2 \frac{\text{cm}^2}{m}, \qquad eq - 7.12.2.1 \ ACI\ 318 - 08$$

Select $\emptyset 14\ mm$ with $\text{As} = \frac{1.4^2 \times \pi}{4} = 1.54 \text{ cm}^2$

$n = \frac{7.16}{1.54} = 4.9 \approx 5\emptyset 14$ bars for 1 meter width

$\therefore use\ \emptyset 14@20\ cm$

check maximum spacing :

$$s = 380\left(\frac{280}{f_s}\right) - 2.5c_c = s = 380\left(\frac{280}{280}\right) - 2.5 \times 50 = 25.5 \ \text{cm} > 20cm \rightarrow O.K$$

5- Design for Y-Direction Positive Bending Moment (Top reinforcement) :

Mxx = 41.3 KN.m/m

$$\text{Kn} = \frac{\frac{M_u}{\emptyset}}{bd^2} = \frac{41.3 \times \frac{10^6}{0.9}}{1000 \times (334)^2} = 0.41 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho_{required} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 0.41 \times 20.6}{420}}\right) = 0.001$$

$$A_{s\,required} = \rho_{required} \times b \times d = 0.001 \times 100 \times 33.4 = 3.3\ cm^2/m$$

$$A_{s\,minimum} = 0.0018 \times 100 \times 40 = 7.2\ \frac{cm^2}{m}\ , \qquad controls$$

<u>Select Ø16 *mm*</u> with $As = \frac{1.4^2 \times \pi}{4} = 1.54\ cm^2$

$n = \frac{7.2}{1.54} = 4.7 \approx 5Ø14$ bars for 1 meter width

$\therefore$ *use Ø14@20 cm*

<u>check maximum spacing :</u>

$$s = 380\left(\frac{280}{f_s}\right) - 2.5c_c = s = 380\left(\frac{280}{280}\right) - 2.5 \times 50 = 25.5\ cm > 20cm \rightarrow O.K$$

Table (4-12): Base Slabs Reinforcement Results

Base slab	Thickness	X-Bottom Steel	X-Top steel	Y-Bottom Steel	Y-Top steel
Level -1.5	40 cm	Ø14@20cm	Ø14@20cm	Ø14@20cm	Ø14@20cm
Level -1.5/inclined	40 cm	Ø14@20cm	Ø14@20cm	Ø14@20cm	Ø14@20cm
Level -3	40 cm	Ø16@15cm	Ø16@15cm	Ø14@20cm	Ø14@20cm
Level -3/inclined	40 cm	Ø16@15cm	Ø16@15cm	Ø14@20cm	Ø14@20cm
Level -6	40 cm	Ø16@15cm	Ø16@15cm	Ø14@20cm	Ø14@20cm

4.4 Design of Cafeteria's Shell

Shells as a Structural Form

Thin shells are an example of strength through *forms* as opposed to strength through *mass* . The effort in design is to make the shell as thin as practical requirement will permit so that the dead weight is reduced and the structure function as a membrane free from large bending stress . by this means , a minimum of material is used to the maximum structural advantage.

A shell structure is a thin curved membrane or slab usually of reinforced concrete that functions Both as structure and covering .The term "shell" is used to describe the structures which possess strength and rigidity due to its thin, Natural and curved form such as shell of egg, a nut, human skull, and shell of tortoise.

The Shell Shape

The shell used to cover the cafeteria building is double curvature shell that is formed by the translation of tow unequal concave down curves placed at right angle to each other . our shell can be classified under ***surface of translation*** , ***synclastic*** , ***non-developed*** shell:

Surfaces of translation : are generated by sliding a plane curve along another plane curve, while keeping the orientation of the sliding curve constant.

Synclastic shell: these shells are doubly curved and have a similar curvature in each direction.

Developable surface : is a surface that can be unrolled onto a flat plane without tearing or stretching it.

4.4.1 Code provisions

the following provisions are taken from "Concrete Shell Structures Practice and Commentary" ACI 334.1R-92 (Reapproved 2002) in combination with ACI 318-08 (chapter 19 - SHELLS AND FOLDED PLATE MEMBERS)

1- Shell Analysis

• the shell could be analyzed using various method such as classical shell theory, simplified mathematical or analytical models, or numerical solutions using finite element, finite differences, or numerical integration techniques. In our case , due to the uni-symmetry and complexity it is not possible to derive a mathematical equation to find the internal forces .Instead , we will use the *Finite Element Method* .

• Elastic behavior is the commonly accepted basis for determining stresses, displacements, and stability of thin shells.

2- Shell thickness

• Shell thickness is not always dictated by strength requirements but often by deformation of edge members, stability, and cover over reinforcing steel.

• Stress concentrations due to abrupt changes in section shall be considered and, where necessary, the thin shell shall be gradually thickened.

3- Shell Reinforcement

• Shell reinforcement shall be provided to resist tensile stresses from internal membrane forces, to resist tension from bending and twisting moments, to limit shrinkage and temperature crack width and spacing, and as reinforcement at shell boundaries, load attachments, and shell openings .

• Reinforcement for shear and bending moments about axes in the plane of the shell slab shall be calculated in accordance with Chapters 10, 11, and 13 in ACI 318M-08.

4- Minimum amounts of steel

• in tensile zones, a minimum area of 0.35% in each two directions longitudinally and transversely for crack control.
• in compression zone, a minimum steel that is usually provided in slabs, 0.18 % percent of steel.

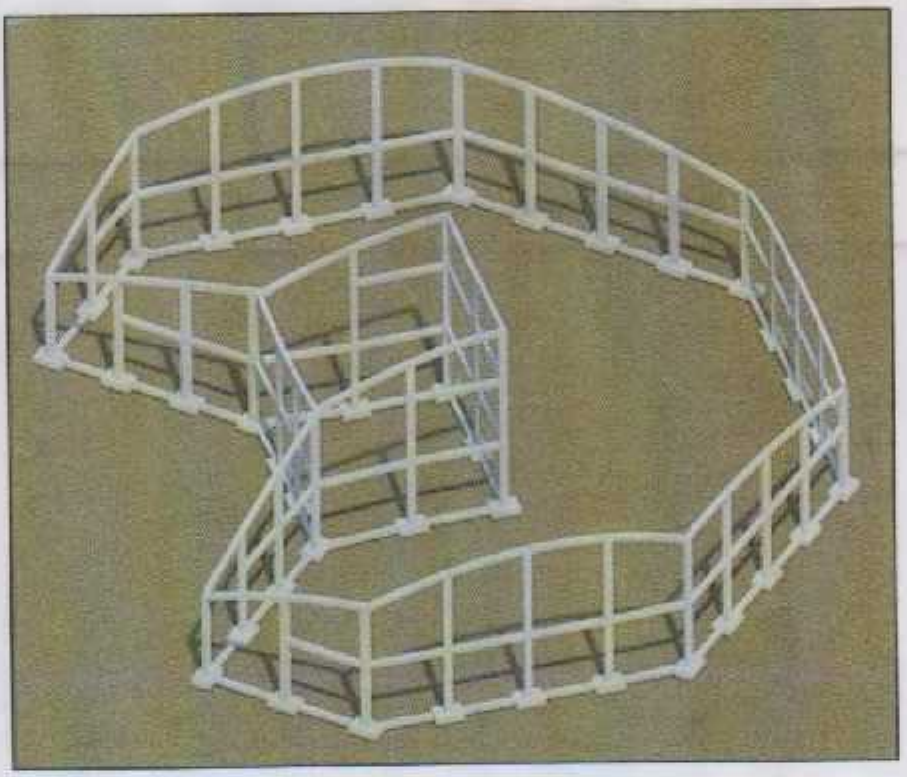
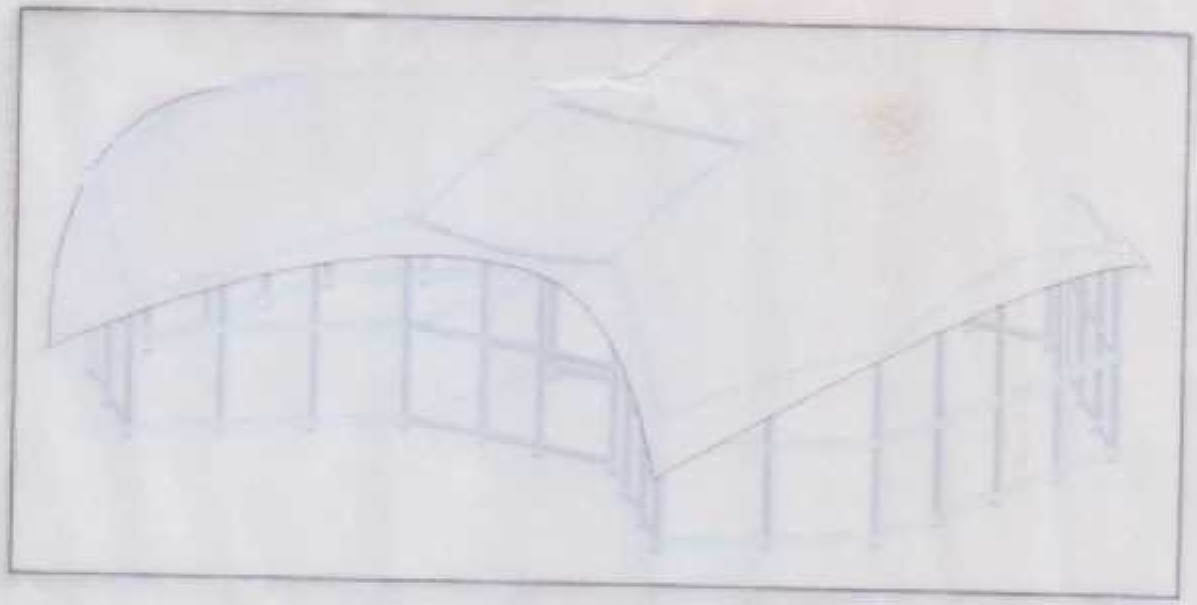


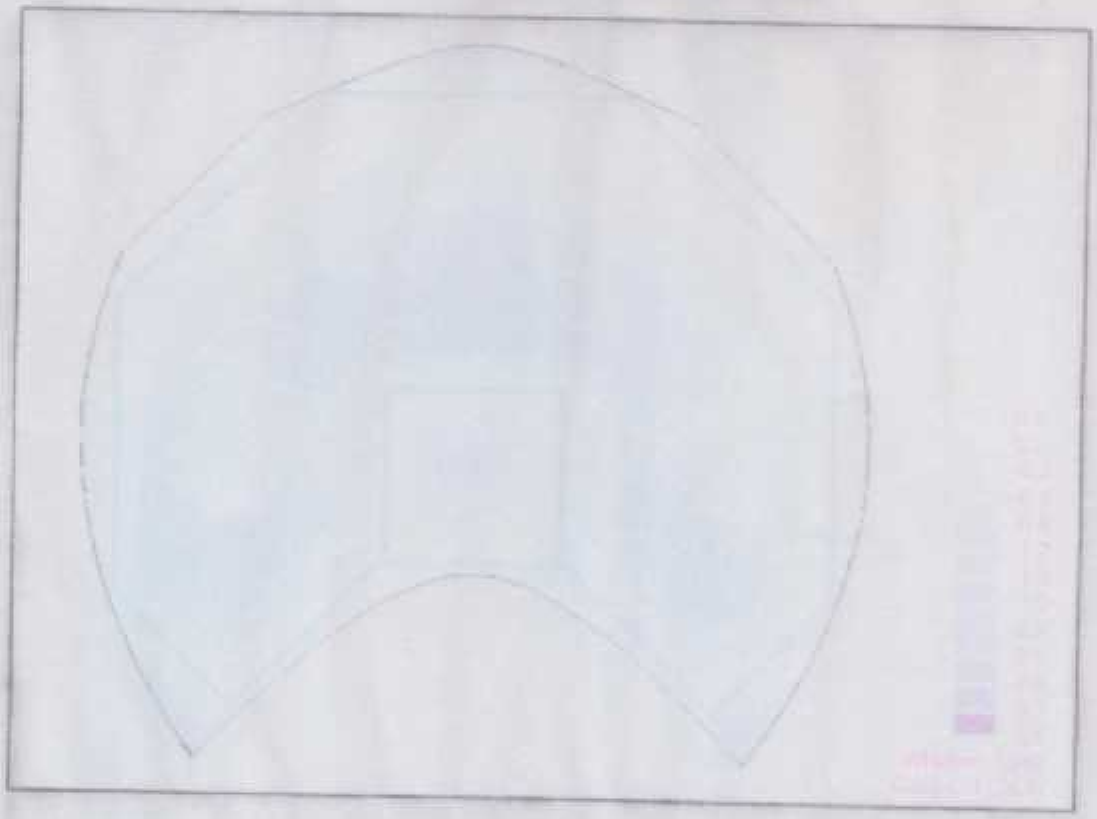
Figure (4-92): **Support System of the shell**

The support system consists of concrete curved beam directly under the shell supported be concrete columns .

4.4.2 Structural Analysis of the shell



Analytical model of shell



Deformation Result due to the service own weight

Membrane force in X-direction N_{xx} (negative sign indicate the compression force while the positive sign indicate the tension force)

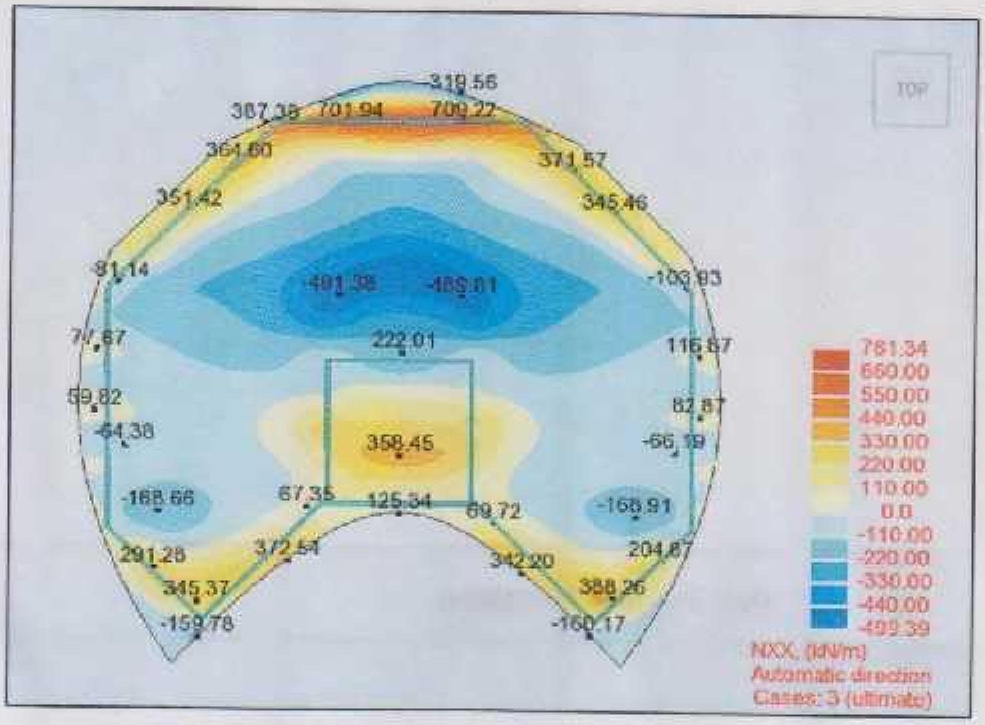


Figure (4-95): Membrane force in X-direction Nxx

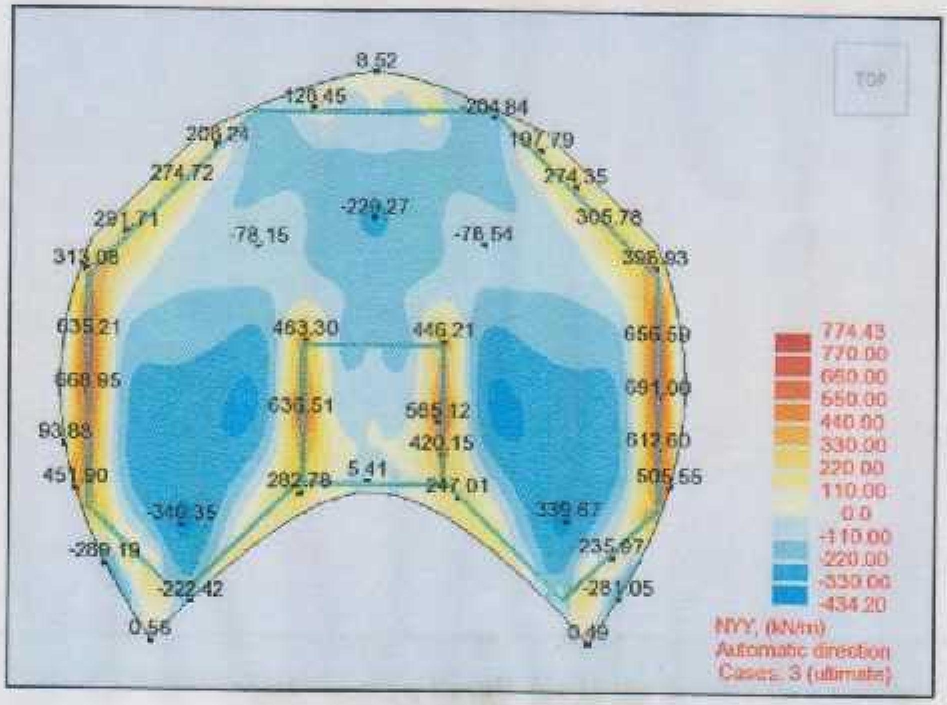


Figure (4-96): Membrane force in X-direction Nyy

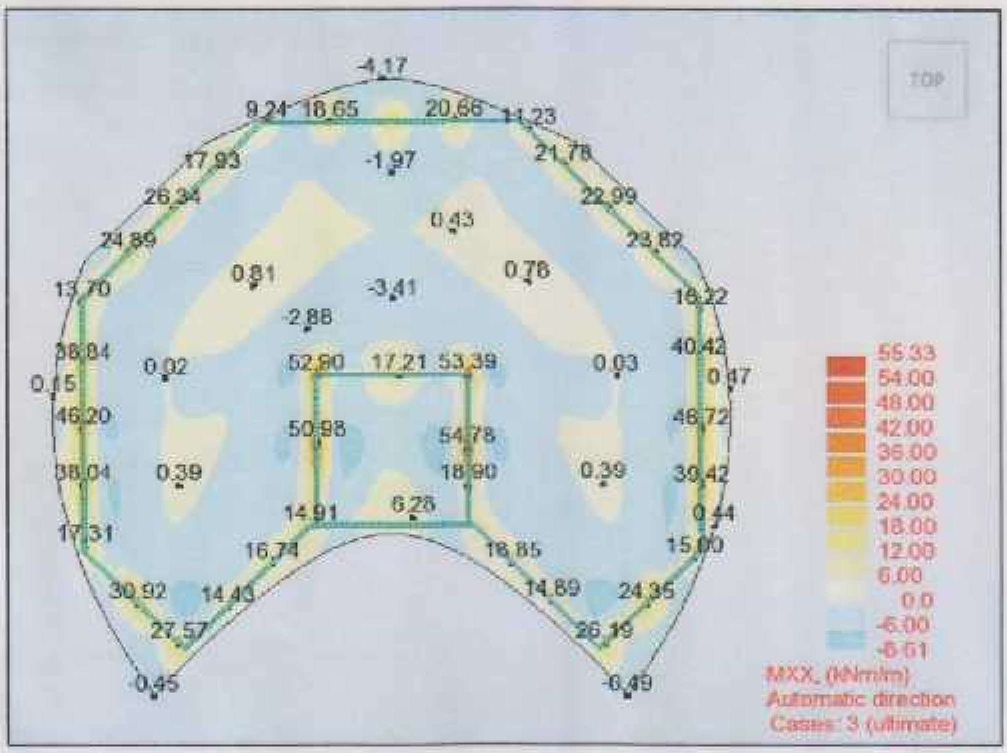


Figure (4-97): Bending moment in x-direction Mxx

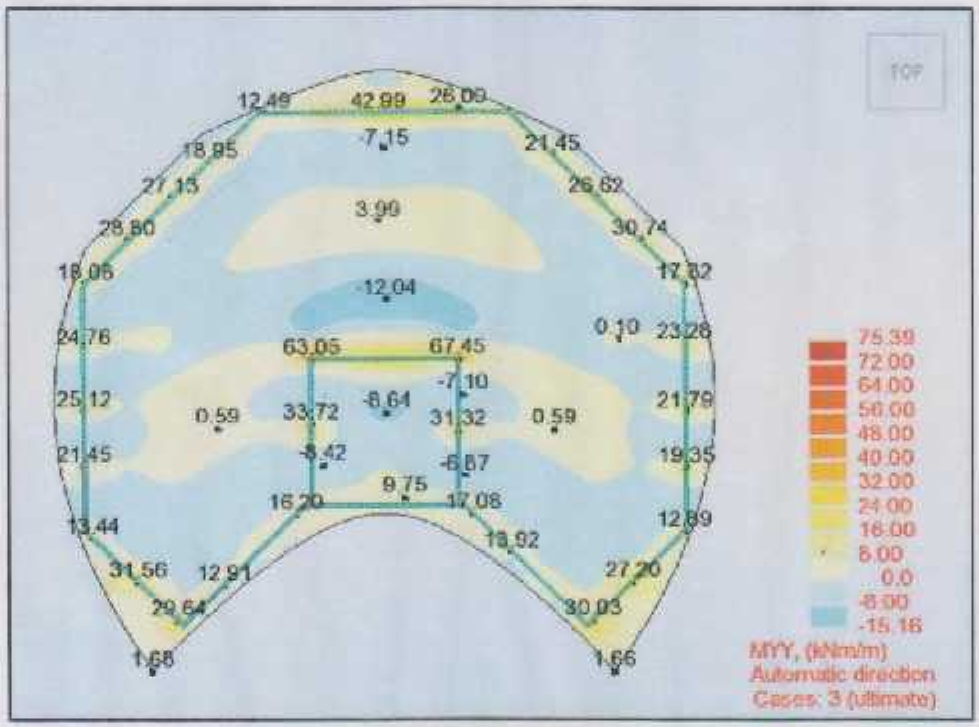


Figure (4-98): Bending moment in y-direction Myy

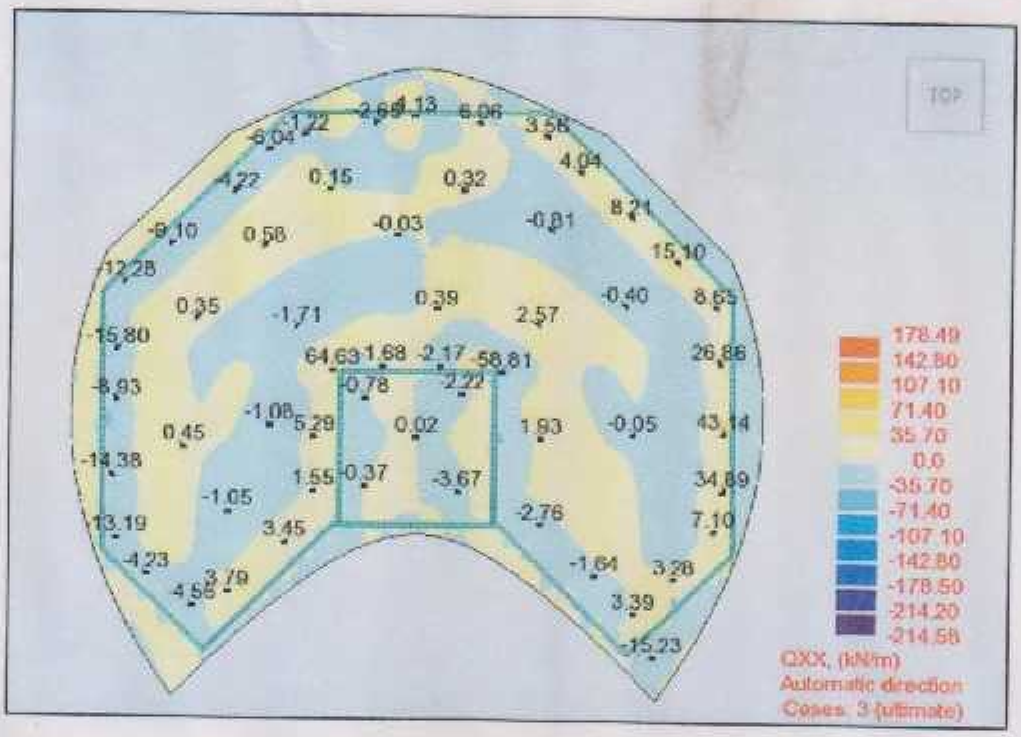


Figure (4-99): Shear Force in x-direction Qxx

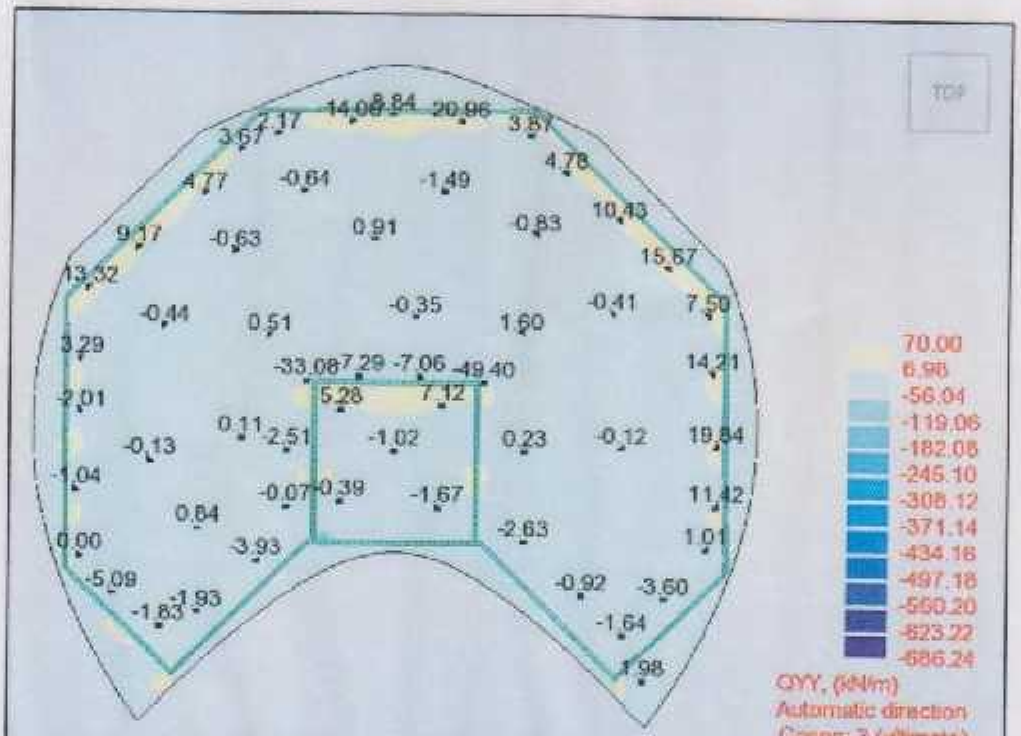


Figure (4-100): Shear Force in x-direction Qyy

It is noted from the above results that the Bending moment and shear forces values are so small so that it can be neglected except above the supports .

4.4.3 Structural design of the shell

Design of Compression Forces :

The maximum compression force is N=494.44 KN/m , the section will be designed as a compression rectangular section with height of 15 cm and 1 m width :

$P_u = \emptyset \times P_n = 0.65 \times 0.80 \times A_g(0.85 \times f_{c'}(1-\rho_g) + f_y \times \rho_g)$

$494.44 = 0.65 \times 0.80 \times 150000(0.85 \times 24(1-\rho_g) + 420 \times \rho_g)$

$\rho_g = -0.031 < 0.0018$ is less than the minimum reinforcement ratio

$A_{s\ minimum} = 0.0018 \times 100 \times 15 = 2.7\ cm^2/m$

Select $\emptyset 10\ mm$ with As $= \frac{1.0^2 \times \pi}{4}$ =0.785 cm^2

$n = \frac{2.7}{0.785} = 3.44 \approx 4\emptyset 10$bars for 1 meter width

$\therefore$ $use\ \emptyset 10@25\ cm$ for both Directions in the .

Design of Tension Forces :

The maximum tensile force in the shell (not above the support)is N=358.45 KN/m . the whole force will be carried be the reinforcement steel only :

$$A_{s\ required} = \frac{N}{f_y} = \frac{358.45x1000}{420} = 8.53\ cm^2/m$$

Not less than :

$A_{s\ minimum} = 0.0035x100x15 = 5.25\ cm^2/m$

$A_{s\ required} = 8.53\ cm^2/m$ Controls

Select $Ø12\ mm$ with $As = \frac{1.2^2 \times \pi}{4} = 1.13\ cm^2$

$n = \frac{8.53}{1.13} = 7.54 \approx 8Ø12$ bars for 1 meter width 4 bars in top and 4 in bottom

$\therefore$ use $Ø12@25\ cm$ for both Directions top and bottom .

Design the shell above the supports:

Design of shear forces :

The maximum shear force in the shell above the support is Q=64.6 KN/m , the force is designed so that no shear reinforcement is required :

$d = 150 - 20 - 10 = 120$ mm

$Ø \times V_c = Ø \times \frac{1}{6} \times \sqrt{f_{c'}} \times b_w \times d = 0.75 \times \frac{1}{6} \times \sqrt{24} \times 1000 \times 120 = 73.5$ KN

$Ø \times V_c = 73.5$ KN $> Q_{xx} = 64.4\ KN$

Design of Bending Moment :

The bending moment direction that appears above the supporting beams is perpendicular to them.

the maximum Bending moment is M=64.6 KN/m :

$$Kn = \frac{\frac{M_u}{Ø}}{bd^2} = \frac{64.6 \times \frac{10^6}{0.9}}{1000 \times (120)^2} = 4.97 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho_{required} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 4.97 \times 20.6}{420}}\right) = 0.0138$$

$A_{s\ required} = \rho_{required} \times b \times d = 0.0138 \times 100 \times 12 = 16.55\ cm^2/m$

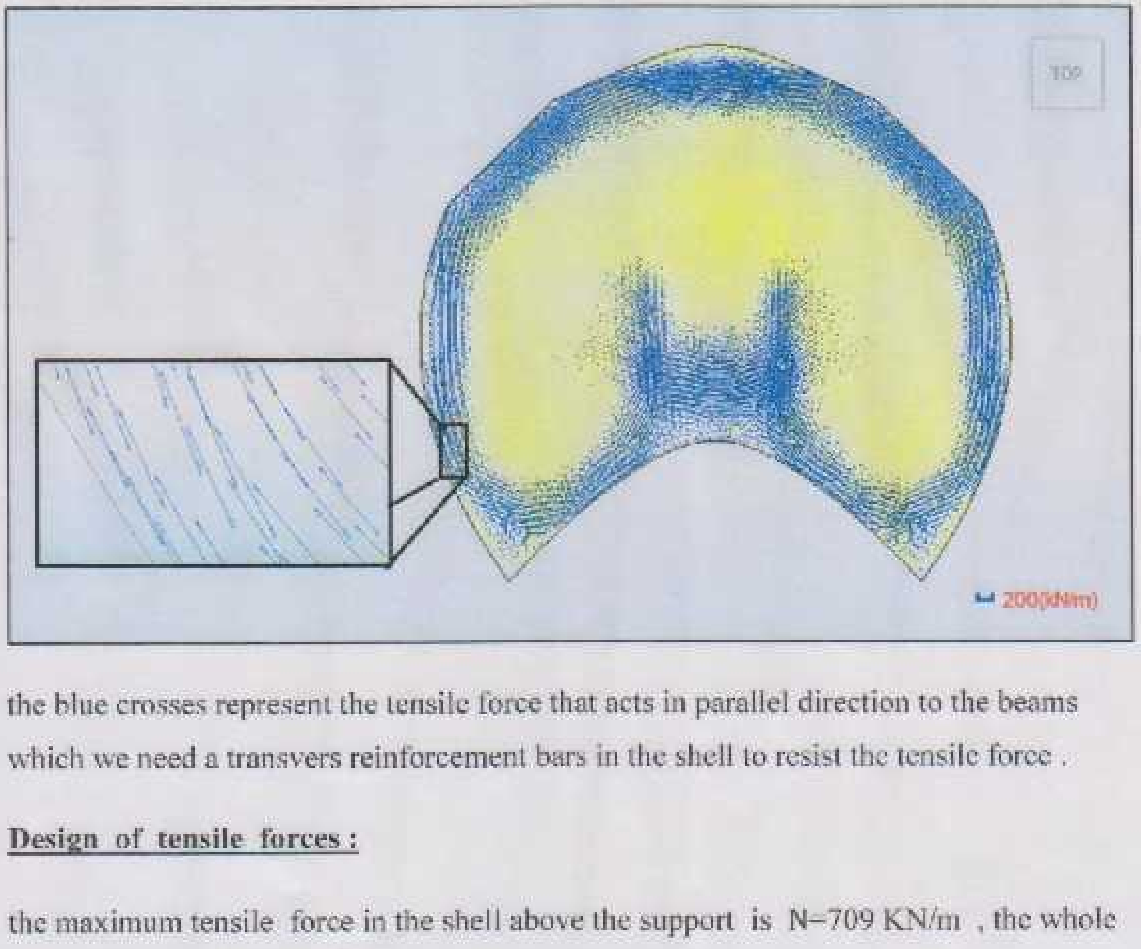
$A_{s\ minimum} = 0.0035x100x15 = 5.25\ cm^2/m$

Select $Ø12\ mm$ with $As = \frac{1.4^2 \times \pi}{4} = 1.54\ cm^2$

$\therefore$ *use* Ø14@9 *cm* with As=17.11 cm^2/m in longitudinal Directions perpendicular to the supporting beams .

Design of Tension forces :

To indicate precisely where is the tensile force acting and where its direction , we used the crosses presentation from the software :



the blue crosses represent the tensile force that acts in parallel direction to the beams which we need a transvers reinforcement bars in the shell to resist the tensile force .

Design of tensile forces :

the maximum tensile force in the shell above the support is N=709 KN/m , the whole force will be carried be the reinforcement steel only :

$$A_{s\,required} = \frac{N}{f_y} = \frac{709x1000}{420} = 16.88\ cm^2/m$$

Not less than :

$$A_{s\,minimum} = 0.0035x100x15 = 5.25\ cm^2/m$$

$$A_{s\,required} = 16.88cm^2/m \text{ Controls}$$

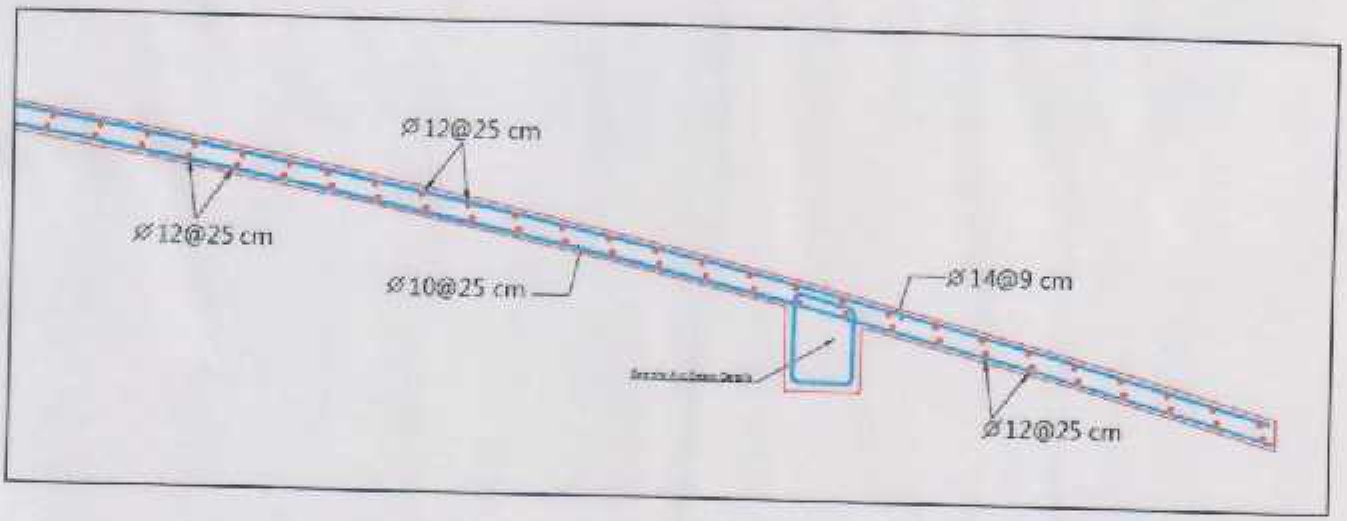
Select $Ø12\ mm$ with $As = \frac{1.2^2 \times \pi}{4} = 1.13\ cm^2$

$n = \frac{16.88}{1.13} = 14.9 \approx 15Ø12$ bars for 1 meter width

$\therefore$ *use Ø12@5 cm* for the transvers Directions .

Reinforcement Detailing

The reinforcement steel that resists the tension and compression stresses in the middle of the shell (not above the support) is placed in the bottom and top layers of the shell. However, the steel that resist the bending moment is place in the top layer perpendicular to the steel resisting the tensile force that acts along the beams.



4.4.4 Design of Arc-Beams supporting the shell

The beams supporting the shell are subjected to bending moment and large tensile axial force , so the beam dimensions is 40 cm in width with 50 cm in height . The design is carried out for all the beams span by span but we will show Arc-beam 3 (particularly the middle span) as an example :

Analysis Results of Arc-Beam 3:

Figure (4-101): Shear forces diagram of Arc-beam 3

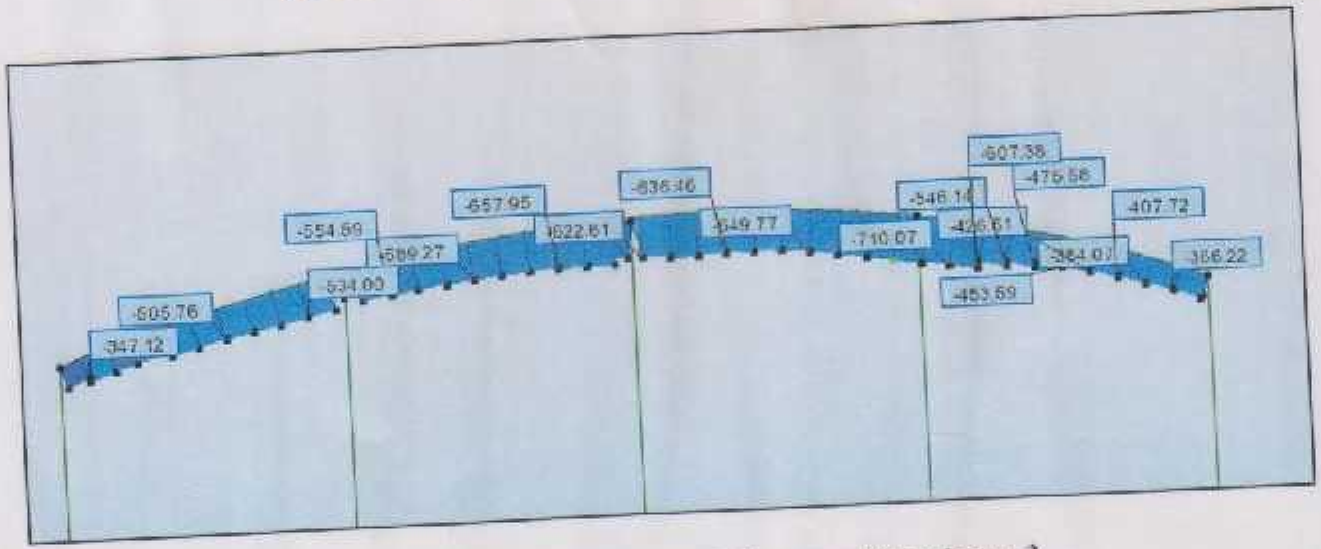


Figure (4-102): Tensile forces Diagram of Arc-beam 3

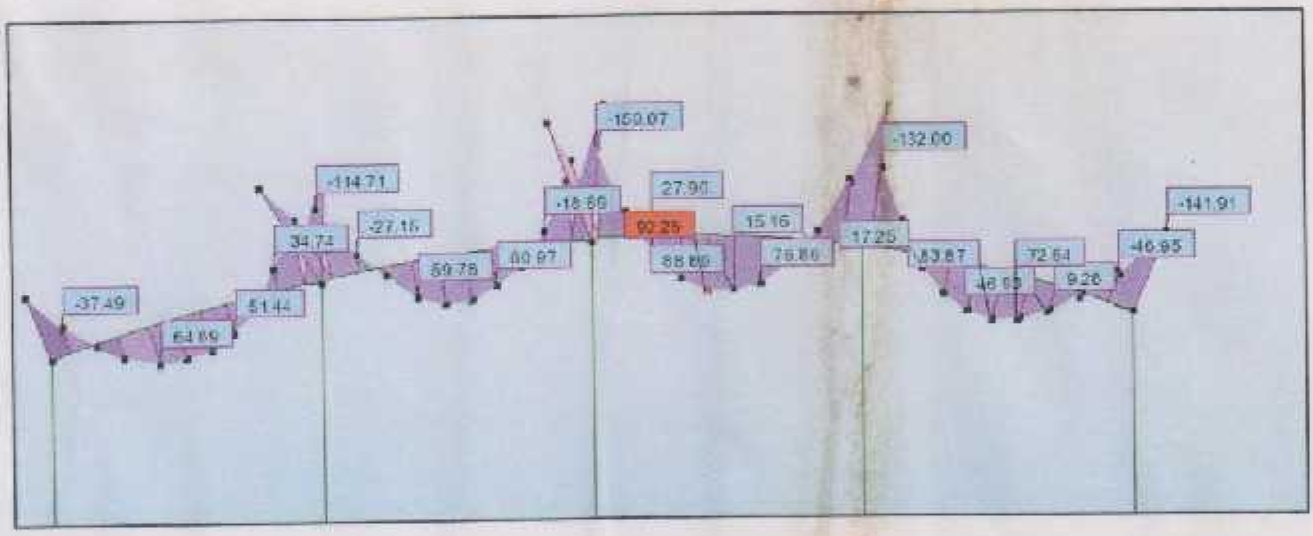


Figure (4-103): Bending Moment Diagram of Arc-beam 3

Design of Shear Forces:

the shear force at distance (d) from the face is V_u=267.57 KN and the tensile force

$N_u = 649.77$ KN

Vc for member subjected to significant axial tension is given by 11.2.2.3 ACI-318-08

$$\emptyset \times Vc = 0.75 \times 0.17\left(1 + \frac{0.29 \times N_u}{A_g}\right) \times \sqrt{f_{c'}} \times b_w \times d$$

where Nu is negative for tension and Nu /Ag shall be expressed in MPa

$d = 500 - 40 - 10 - 8 = 442$ mm.

$$\emptyset \times Vc = 0.75 \times 0.17\left(1 + \frac{0.29 \times -649770}{400 \times 500}\right) \times \sqrt{24} \times 400 \times 442$$

$\emptyset \times Vc = 6.38$ KN the concrete shear strength is very small , shear force is carried by the steel :

$$Vs = \frac{Vu}{\emptyset} + V_c = \frac{267.57}{0.75} - 6.53 = 350.23\ KN$$

$$\frac{Av}{S} = \frac{V_s}{f_y \times d} = \frac{350.23 \times 1000}{420 \times 442} = 1.89\ , Av = 2 \times 113 = 226\ mm^2$$

$$s = \frac{226}{1.89} = 119.6\ mm$$

Select Ø10 mm stirrup @10 cm spacing

Design of Tensile forces:

the magnitude of tensile force is $N_u = 649.77$ KN , the whole force will be carried be the reinforcement steel only :

$$A_{s\,required} = \frac{N}{f_y} = \frac{649.77x1000}{420} = 15.47\ cm^2$$

Select Ø16 mm with $As = \frac{1.6^2 \times \pi}{4} = 2\ cm^2$

$n = \frac{15.47}{2} = 7.73 \approx 8Ø16$ bars

4 bars in the top and the other 4 bars in the bottom.

Design of Bending Moment:

the magnitude of negative Bending Moment at the face is $M_u = 148.6$ KN :

$$Kn = \frac{\frac{M_u}{Ø}}{bd^2} = \frac{148.6 \times \frac{10^6}{0.9}}{400 \times (442)^2} = 2.11\ Mpa$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho_{required} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 2.11 \times 20.6}{420}}\right) = 0.0053$$

$$A_{s\,required} = \rho_{required} \times b \times d = 0.0053 \times 40 \times 44.2 = 9.4\ cm^2$$

$$As_{min} = 0.25 \times \frac{\sqrt{f_c'}}{f_y} \times b \times d = 0.25 * \frac{\sqrt{24}}{420} * 400 * 442 = 515.56\ mm^2$$

Not less than

$$As_{min} = \frac{1.4}{f_y} * b * d = \frac{1.4}{420} * 400 * 442 = 589.33\ mm^2$$

$As_{req} = 940 mm^2 > As_{min} = 589.33\ mm^2$

$$n = \frac{9.4}{2} = 5$$

Select 5Ø16 in the top

the Positive Bending Moment in the middle span s $M_u = 92.2$ KN :

$$Kn = \frac{\frac{M_u}{\emptyset}}{bd^2} = \frac{92.2 \times \frac{10^6}{0.9}}{400 \times (442)^2} = 1.31 \text{ Mpa}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{420}{0.85 \times 24} = 20.6$$

$$\rho_{required} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2.K_n.m}{f_y}}\right) = \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 \times 1.31 \times 20.6}{420}}\right) = 0.0032$$

$$A_{s\,required} = \rho_{required} \times b \times d = 0.0032 \times 40 \times 44.2 = 5.71 \ cm^2$$

$$As_{min} = 0.25 \times \frac{\sqrt{f_c'}}{f_y} \times b \times d = 0.25 * \frac{\sqrt{24}}{420} * 400 * 442 = 515.56 \ mm^2$$

Not less than

$$As_{min} = \frac{1.4}{f_y} * b * d = \frac{1.4}{420} * 400 * 442 = 589.33 \ mm^2$$

$$As_{req} = 571 mm^2 < As_{min} = 589.33 \ mm^2$$

$$n = \frac{5.89}{2} = 3$$

Select 3Ø16 in the Bottom

References

- Building Code Requirements for Structural Concrete (ACI-318-08), USA, 2008.

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