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BRIDGE DESIGN (TEORIA)

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- S. Angelo Bridge → use with nowadays traffic even if it wasn't projected for this. 3 arches are original the others were rebuilt with same shape and material. It works in ≠ situation: flow of river change as the speed, pollution...

- Regensburg bridge typical shape of long bridges of Middle Ages and Europe. Each arch is ~ 20m. Perfectly conserved.

- Pont St. Benezet. Only 4 arches. There is a small chapel. It was quite long → 22 arches → big arch → problems arches big, span big and flow of river is strong → collapse two times.

- Scaligero bridge → span <
  masse is hudge
  the design is worse than the previous.

- Karol bridge: arches span of 30 m
  protection of wood from flow
  heavy piers and arches
  proportion of masses

- London bridge (no more exists) the longest of XII. Thickness of piers → narrow gaps between the piers to let the water pass through. This idea was connected to the use of the stone and because the high weight of bridge → section reduced and speed of the water under the bridge increases of 2/3 times so if you have a fluid with a big acceleration close the piers, behind the piers you have a section effect → takes away material from under the pier: fluid accellerating going under the piers and then decellerating after piers → vortex on the back sides of the piers and the ground is taken away under the piers to the valley.
  It isn't demolished for hydrologic troubles but because it was a bridge with house on it. Mid. Ages lot of people lived here = hygiene problems → in XIX a lot of areas were rebuilt.

- Ponte Vecchio: bridge with houses → same reason → city was growing very quickly so they decided to made houses even on the bridge

- Paris other ex of b. with houses. Demolished.

- Roman Bridge in Mostar rebuilt in 2004. Built in 1500 on roman ruins we didn't have roman ruins, we know this from hystoricians → was built in Middle Age with roman shape.
  The arches are generally full: there are 2 open section → thickness of arch and the part below is the real arch

- Lavertezzo: on roman ruins and design the upper part of the bridge is not straight → it matter of purpose → the weight of the material is calculated in such a way that the arch is working ~~fun~~. The idea is to have a variable depth → kept ~~the stre~~ compression in the arch constant
  Arch bridge are working here abutments of arches are strong → built arch bridge on the rock → horizontal component of the stress that is taken away by the fundation

- Kintai → Japonese tradition: wood → not a lot of historical bridges - destroyed by fire. Arch slender and light

- Ponte degli Scalzi → biggest arch in Canal grande if don't consider Calatrava bridge. Calatrava bridge problems = difficult in construction → because foundation ground in Venice is very poor and ~~there~~ there is an horizontal component of reaction is hudge and taking away it without piers is not easy and so a particular attention to fundation ②

11/03/08

BRIDGE DESIGN

- Historical notes on bridge construction -

~~1 age~~ Divide the history of bridges in 3 age:

1° age beginning of hystory (ancient romans, egyptians) ~ 1800 → long age → tecnology changed not only for bridges

2° age: beginning of ~~XIX~~ century to the beginning of XX century → INDUSTRIAL REVOLUTION period in which old materials (stones, bricks, woods) substituted by IRON.
The trasport by rail developed and bridges changed because loads of rail trasport were completely different from the road transport. (pedestrian, animal...)

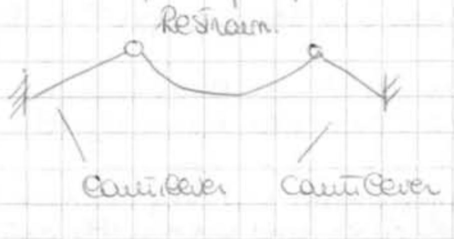
3° age XX century - nowadays New tecnology and everything we know about bridges is connected to this age.
Nowadays no one builds a bridge with a static scheme which comes from the II age.

I AGE

Transport made by using animals and bridges were used for crossing rivers and other natural obstacles.
The material used were simple and natural → WOODS (trunks, beams, wooden deck)
→ NATURAL FIBERS (hemps)
→ STONE more important.

Pg 4 Bridges handmade

① Nowadays bridge of Nepal built by hemps



STATIC SCHEME

The bridge is made by 2 cantilevers which start from the ABUTMENTS on the ~~sides~~ two sides of the river and on the 2 cantilevers there is a SIMPLE SUPPORTED BEAM.
The restrain ~~of~~ in the abutments is given by the weight of the stone that is used to keep in place the cantilever

② Bridge made by bamboo simple static scheme with cantilevers

Small bridges made by hands not easy

Pg 5

① Bamboo frame on a shallowed river. Structure: lots of bamboo directly fixed to river bed and on them vegetables

② Bamboo → static scheme → cage works as a 3D beam

Pg 6

Conceptual evolution of bridges the schemes for material → static scheme. (realist / natural / their piece)
Here truss static scheme → more efficient → working for long static scheme. (piece)
If we have textile material likes hemps → working in tension. You can build big bridges even not quite long.

①

~~depth~~ concrete works in compression.

- Humber suspended bridge → • deck in steel
  • construction technique: building the piers, then connecting the piers with suspension ropes and lifting the segments of the deck using ropes as slings.

  • section shaped to be efficient for wind → main problem is the interaction between bridge and wind. Wind coming in a transverse direction → see the deck as a wing and on the back of the deck there are vortex and others interaction with the aerodynamic flows → deck move → frequency of wind and his inner frequency
  ↓
  collapse of suspended bridge.

- Millau viaduct → combination of cable stay and box section continuous beam.
It's made for highway, crossing a valley that has a very depth slope.
Without bridge → go down valley → big slope → long a lot of km → crossing directly the valley quickly
Environmental impact: piers taller than tour Eiffel and span 342 m → Environmental impact: deck tall
Piers built in high strength concrete up to the level of the deck then you consee temporary piers made in steel, the deck without the stay cable and the upper of the piers pushed on the piers and the temporary piers so during the construction of the deck, the deck itself in the box section has a pier in collaboration. There is a construction phase in which the deck was done, the upper part of piers (steel) → very light because they carried by lorry special trucks on the deck that was done temporary piers and when the upper part of the pier reached the position it was lifted connected to lower part and then the stay cable is taken place.
Stay cable only for traffic load and permanent actions.
Self weight of deck is carried by the deck alone and not only the self weight but also the weight of this element were put on the deck.

– Basis of design –

Choose the typology of bridge → I know place, kind of road, hydraulic aspect (most of river) but no one tell me how kind of bridge. → fondamental step = no theory, equation says type of bridge = background take best solution to have little cost, strong/durable structure quick speed of construction.

Idea is deeply connected to material. Design is a mixture of idea, abstraction, model and connection of model to real life.

INPUT DATA

- Planimetric configuration is given because of traffic restrict, the design of road or railway give the line where the track has → you don't choose the geometry of your bridge from a planim. config. → you have imposed from traffic limits.
Layout...
Several types the obstacles is not crossed in an easy way. ~~...~~ the best way is to cross normal to axis of the valley but in the most cases the axis ~~...~~ is not ⊥ to valley → cross the obstacle with a direction that is not the best, from the efficiency and services of whole system.

≠ vibration period.
Traffic high frequency vibration
Wind low " "

~~Speedness~~ Stiffness and displacement are connected to these 2 ≠ vibrations.
People walking on bridge frequency 0.5 / 2 Hz
↑ Walk 0.5/1 ↑ Run

Crowd = 1 Hz

Highway trains << 0.5 Hz
Frequency of structure = 0.5/5 Hz × traffic stress.

Wind → period few seconds - lower and very big deformation (especially suspended bridges → transverse deformation of meters (0.5/1.5 m).
Maybe move with period of 20/30 seconds (very slow).

4

Viaducts = obstacle = land → horographic restraints and functional requirements. Restraints are existing infrastructures that have to be crossed → viaducts cross road, railways that are below

= geological and geotechnical most V. are in urban zone → foundations must be built inside urban environment without damaging existing building. Building of. is invasive operation → not use some technology → avoid vibration, tube lines...

= landscape requirements = viaducts can be long, big (km) maybe they are in a zone with beautiful landscape → preserve for aesthetic.

- Service class. In Italy and Europe b. are divided in:

- road
- pedestrian
- railway
- channel
- mixture of all of them

Generally road b. is also a pedestrian b. in some areas → footpath on 2 or 1 sides, except tracks.

Road b. are divided in 2 categories < 1, 2 according to traffic expected to b.

1° category → small roads connected villages on hill, small traffic expected
2° " → b. connected highway → a lot of traffic.
This division was used in the past for design. Usually now build b. on 1 class (heavy traffic). But existing b. (20 years ago) were built according to this subdivision → if I have to make maintenance works, repair the b. for ex of a II class as it was designed for a low load
For 1 new b. built, there are 4/5 old b. to repair → business is on repair. So the technologies connected to repair structures are more useful than capabilities of design new structures
Highway, railway → repair b. → without closing (can't close an highway) → repair with traffic open → when you build a new bridge you have to keep in mind that the new bridge should be designed in such a way, in a simple way so when you have to repair it you can work inside, below, around the b without closing traffic = so you need free resources, extra resistance that will help during repairing

Pedestrian bridges → only for pedestrian traffic = no vehicles are allowed on the bridge = barriers / stairs = impossible to get a vehicle on the bridge except for maintenance options (small vehicles for this).

Railway bridges → passenger traffic ① < ≠ speed/load
mixed traffic ②

① high speed, light
② slow, heavy loads

Channel bridges: transfer water or other installations bearing b → pipelines, aqueduct, gasduct.

Mixed bridges: both road and railway running parallel → you have to respect the requirements of both the categories

Deck b = traffic is going on top of deck
Trough " = " " inside of deck

⑤

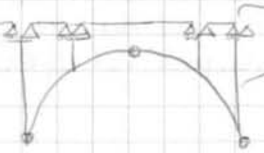
U shaped (through b.) → road inside of U
Truss system iron b = iron cage and traffic is running in the cage

in early morning -> 1 train moving and stopped = mesure.
Sun went up = II train enter the b. = displacement = expected b. moving down
-> wrong effect of temperature bigger than weight of train and b. moved up because the upper part was hot, while the lower was cold -> iperstatic effect. -> smaller deflection with 2 train.
Effect of T. is related to stiffness = strong b. -> hudge external action due to T.
flexible " -> small " " "

ARCH BRIDGES ≠ static scheme from point of view of arch

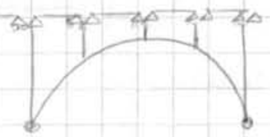
• 3 hinges arch

isostatic



• 2 hinges arch

iperstatic
no hinge at top



Arch can be separated from deck or touch.

• Bow string The deck is below of the arch, good solution for static scheme because H action coming from arch (give trouble to found.) in this case H action are taken away by deck that it is the STRING (CORDA), whereas the arch is the bow.
The deck is slender because distance of vertical element is small -> lot of vertical element. The deck works with the span of the distance of the vertical elements.
The deck connecting the basis of the arch the structure is self equilibrated = only V actions go to foundations
Arch used for flat landscape -> cross a flat = no anything that take away the H action = so use this arch
The previous arches used to cross valleys because found. are founded in hills or valley so valley provides force to take away the H actions

• Fully restrained arch
Most iperstatic (2 full restraints) used when there is rock = found. in rock = strong and rock provides full restraints
only this found give

ex of arch bridges. pg 14

FRAMED BRIDGES

• Frame with a gerber beam -> 2 ≠ static scheme: the frame, the bearing, full connection of bear and beam = frame and then gerber beam which connects the frame.

• Frame connected with double pendulum -> uncommon solution.

• Continuos frames.

Build a pier from the pier we build two cantilevers one on each side one on the left and one on the right... at the end we have a T shaped (hammer)

1 pier and a part of the deck that is over the pier and there is full connection (full restrain).

(6)

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SUSPENDED BRIDGES

Hudge spans.

- Anchored to the ground — Suspension anchored to ground
  Deck can be continuos or simply supported.

- Self anchored (to deck) — anchored to ground and deck
  Deck is continuos

Idea of suspended b. is old → 1820 → 180m long span for that time = use only iron.

→ ARCH BENT BRIDGES

tendons — steel

Arch solution. Opposite to suspended bridge (vertical elem / arch tensioned).
Deck above, vertical elements are compressed.
(Change material — concrete)

HOST USED CONSTRUCTION SYSTEM

→ Formwork = cassero = made of pannels that act as a mould for designing shape of concrete for any purpose (contain cls in pouring).

→ Falsework = centina = temporary support structure for formwork

- Formwork on fixed falsework → falsework ~~can~~ must be safe (problem during construction)
  → calculate deformation of falsework during casting because the weight of the concrete can deformate the falsework
  The ~~deck~~ deck is inside the formwork supported by falsework
  → generally falsework inside the 2 piers ~~and~~ under control part and the 2 cantilevers of deck.

Falsework on ground.

Construction profile → to cast one beam and a 1/4 (1/5) of the II beam a then leave a coupling joint here then move the falsework in the II span and cast the II part of the II span and a 1/4 of the III span...
You have to build the falsework for one span and 1/4 and you can reuse the same falsework for every span = the length of the span must be the same.
Little money for falsework.
The joint usually is at 1/4 / 1/5 because it's the region in which bending moment of permanent actions is +/- ∅. And in this region there is the coupling of the prestressed tendons used during construction.
Technique used when the distance deck/ground < 10m.
Falsework high = expensive.
Take account to deformation of falsework during casting = falsework ~~flex~~ under weight of fresh concrete deformates = ~~keeps~~ when I put falsework ~~to~~ there is no stress on it, then it's compressed → ~~xxxx~~ after pouring compressed, to ground and I can't move it → if I don't use special screws I can unload.

Falsework sliding
Piers tall → falsew = spans from one pier to other and slides from one pier to other, made of truss system of iron → we have temporary supports fixed to pier
~~xxx~~ There is the completed deck, you place formw for new part of deck = the falsew is

7

Blister = where there are anchorage for prestressed tendons. because they are running inside deck but when you have to prestressed you should have they outside.
So tendons for inside to the deck is coming out in the blister, and in the blister you place the hydraulic jack and you pull the tendons = close anchorage = made the prestressing operation.
In each segments there are blisters where you add new tendons → each segments has its tendons. A lot of holes because you have a lot of tendons → 4 tendons enter the b. in each element (two on each side). The tendons enter, run inside the deck and go out in the segment with the same number on the other side.
Tendons use for segment 4 on one side are the same for segment 4 on other side.
Segment with same n° show same tendons.
Pier element has a lot of holes → crossed by all the tendons of the segments of the one side.
Segment close to pier is full of holes and the segment close to midspan as less holes.

Opposite: midspan lot of holes in the bottom flange.
At end construction → I have 2 hammers near → place continuity tendons that go from one pier to other. Tendons for M<0 during construction in this hammer in upper part but I need M>0 during service life → each segments has tendons going in bottom flange that are common to other hammer, these tendons will enter segments from blister → I don't see holes because they are on other side. These tendons are <0 and running to pier ①, these tendons are >0 and running to pier ② . In midspan we have a lot of holes in the bottom and on pier no because M<0.

Pg. 37 I need a lot of place. Prefabrication scaffolding = elements precast here (heavy transport)
Element precast on ground → store and lift with crane.
I can do that because area accessible → road under bridge = not expensive.
In mountain without road accessible, flat area no possible.
lot of b. made with this technique. span 100 m/110 m h = 40m.
This technique started in France at end of '70, in Italy first viaduct in Udine, and b. of Messina/Palermo, highway Isernia/Castelfranco.

1° application - problem of durability in particular French b. (beginning). After 2 collapse during construction they had been forbidden in Germany, but they were used in other countries
At end of '80 in USA very used → Association ASBI produced recomendation for construction.
lot of application in South America (Venezuela)

Problem = frict. of tendons → joins filled only with boxeries and there isn't ordinary reinforcement crossing the joints but only tendons → work also on caracteristic of tendons → joints should be compressed. If you have more friction than expected → having tendons compressed → open and then the risk of corrosion ✱
Join weak point = protected by residual compression stress.
Accident due to typically for span of cantilever → starts from pier and mount segments → problem of disequilibrium → deck connected to pier by temporary tendons.
External tendons allowed.

Working parallel → foundation, pier using self formin → 3/4 m/day → particular casting in parallel precast segments → mount 6/8 segments/day 3m each = 18/24 m/day.
After mounting = end of construction of hammer, out jacks and inserts. Then tendons

Best system to reduce time of construction.

Pg. 38 → special segment on pier → wall → hole for inspection during life and for movement facilities (machinary for jack → tendons are weight so machinary to move) - Rigid structure because TORSION coming from deck → arrived to pier = TRANSFORM IN A COUPLE OF FORCES that give disequilibrium. With system of strut and tie → resistence.

⑧

Technique ⇒ push the b.
- more than here put in tension tendons and having fix point from wall of the abutments (for planim/altim limitation).

↓ R=cost ↓ R=large

BRIDGE TRANSVERSAL SECTION

Parameters that influence
- Span: connected to statical scheme = I can't pretend a slend deck of sa with span 100m
- Slenderness required (l/h) = high = cost.
- Technology for execution ⇒ in some countries no advanced technology.
- live load
  dead load for dynamic behavior ⇒ railway required not slender (because high load). ⇒ important is deformability.

⇒ SLAB BRIDGES CAST IN SITU

- use for isostatic span < 20m } 25 < depth < 70 cm
  " " continuous " ≥ 30m }

Good solution for crossing skewness
- variable shapes

Slenderness R.C l/H = 15/22
P.C l/H = 18/30

⇒ VOIDED SLAB

Attention to durability ⇒ if you put some light elements = holes = seal them ore close and no water enters → ice!
Other problem: anchored to bottom ~~of~~ formwork otherwise = floating.

Using variable thickness ⇒ keep attention to slenderness ⇒ for aesthetic = not best solution

point is fixed / glue

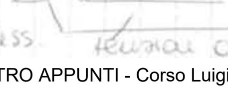
⇒ T-BEAM CAST IN SITU BRIDGES

Mono-beam = when you arrive on abutments → torsion → not take torsion with a single beam ⇒ you need transverse beam ~~...~~ → traverse torsion from the core section to transverse beam

Multibeam = also here transverse beam located on abutments to transform torsion in transverse bending. or internal ~~of~~ to deck ⇒ more complicated.

Slenderness $l/h \cong 20$ (usually 18) → road bridges
" 14 → railway

⇒ INVERTED T-BEAM for channel
For transporting water, or if you have small depth under bridge.
Appearance is very heavy, bad for M>0
For static reason you need a compressed core very large in concrete and a tensile supported steel thin. This is the opposite



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SLAB BRIDGES

Best solution for skew b. or IRREGULAR SHAPE

1) **Massive slabs** with orthogonal edges

- internal action → 5 components: 2 bending moments, 1 torsional, 2 compon. of shear.

Generally with slab theory (rectangular slab) → close resolution

or

- Beam theory with a safe approssimation → imprint of wheels crossing ↑ imprint → design slab with a equivalent beam with same width. → problem transverse bend. mom., beam only longitudinal b.m.
Use such approach: I'll design in transverse direction a T.b.m = 25% L.b.m.
→ in this way maximum moment. → safe.
Possible if slab rectang.

- Concentrated loads → Use influence surfaces or FEM → transposition on influence lines of beam.

Generally I use FEM.

- Generally shear isn't critical → stirrups not necessary. Any case, it should be provided the minimum of value $A_{sw,min}$ given by the code

Small span (<15m) → R.C (no prestressed) → reduce distance between bars to control opening crack → <15cm → for sle.

Span l>15m → P.C → small tendons prestressed at 25/40cm. Ducts for prestressed tendons ϕ=8/10 mm
- to be sure concrete works well with tendons
- avoiding dispersion of prestressing.

2) **Voided slabs**

Compromise: evaluation of internal actions. Take into account voids? No → unique omogeneous structure. Problem transverse direction = flow stress in longitudinal direction consider the massive beam without holes, but in transverse direction stress can't cross holes → introduces massive regions to interrupt the voids →

Having voids → implies an increment of shear → tangential stresses in vertical plane can't cross the voids → all effects reduced → tangential action below → stirrups are necessary because they work like beams.
Transverse reinforcement is generally realized by a double mesh.

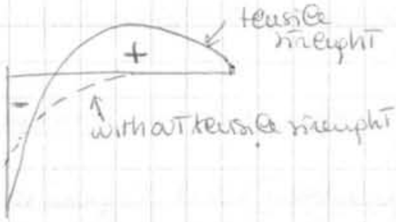
4 bars confined by stirrups.

Generally transverse prestressing bars aren't used because span is small → used in massive regions where tendons should stay inside stirrups.

In corner discontinuity of moment → torque moment along the border near the bearing
Attention = you can crack slab = reduce stiffness and other moments increase → no resist.
Statical scheme ≠ → slab theory for definition of m. → bearing works = if they don't work → uplift = solution no more right. → Moment changes.

What happens to distribution of reactions along border? Distribution of reaction is continuous → there is concentration of action in the corner → uplift of corner region → uplift not acceptable without prestressing → to avoid = bearings discontinuous or capable of elastic deformation

bearings not in steel but in rubber = give deformation. Rubber bearings distribution of reactions.



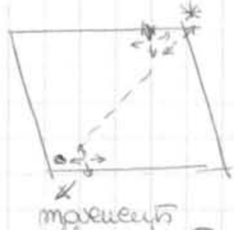
without tensile strenght → not tensile forces = no uplift, reduction of the maximum

Rubber bearings near acute angle (→ stiffness) → preloading to avoid uplift. → avoid to give
bearing in rubber = deformable. → with insert in steel → give possibility to have settlement
Better distribution of reaction.

Typology of bearings.

Block = fixed = no movement allowed

To avoid internal stresses using bearings reduced to min the restraint conditions of plate.
The plate is plain and it's a plate when can control movement of plate, with 3 restraint conditions:



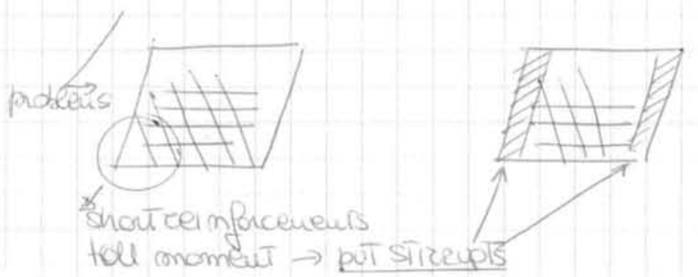
- 2 given by hinge
- rotation about the point → point fixed but all plate can rotate around the point *

To avoid plus along the connection (--) we put a bearing with possibility of trans in this direction so it's impossible a rotation around point because the transverse movement is not possible

All other bearings are free to allow movements

Reinforcements

$\varphi < 0$ longitudinal reinforcements ⊥ to bearings, (vertical with stirrups), transverse reinf. with small inclination respect bearings.



Tem = 5 internal action
Best way but for each element = 5 components of internal action → consider them altogether
Approach = sandwich model → ... when we finished analysis → 5 internal compon. or 8 (shell element)

24/08/'08

- GIRDER BRIDGES -

Solution very used in Italy and the world since the beginning of last century and in. They are used for span of ≈ 35m. Generally they are made in factory so the limit of 35 m derives from possibility of transport → limitation in weight and length

Deck is composed by a certain n° of LONGITUDINAL BEAMS (minimum 3, better 4), some TRANSVERSE BEAMS and IT'S COMPLETED BY SLAB.

MAIN ADVANTAGES:

- We have a TOP SLAB ABLE TO TRANSFER THE LOAD DIRECTLY APPLIED BY TRAFFIC TO THE BEAMS (in this composition of beams the top slab has as first function to receive directly the load of the wheels and transfer them to beam.
- SLAB RIGIDLY CONNECTED TO BEAMS IS THE COMPRESSIVE CHORD IN THE HYPERSTATIC SCHEME OR PARTIALLY COMPRESSED / TENSILE CHORD of the # BEAMS. Because it's participate to Resistance of the single beam.
- IN case of CONCENTRATED ACTIONS like load coming from wheels, it's able to transform the concentrated actions in locally distributed actions and to transfer actions to the beams
- IN HORIZONTAL ACTIONS the TOP SLAB WORKS AS A MEMBRANE IN HORIZONTAL PLANE (very rigid) and then distributed the H forces to the beams.
- Presence of slab increase the LEVEL ARM of the SECTION → consider the beams with the top slab → the level arm is increased due to the fact that the compressed part region is well localized and then the actions are high in section and internal lever arm of the internal couple (the resisting moment) is very high. LEVER ARM

DESIGN CRITERIA

In past very used cast in situ beams → better to reduce the n° of beams. With cast in situ beams, because of problem of formwork and scaffolding, it's better to have the minimum number of casting sections and then we reduce as much as possible the number of beams. On opposite if we use, like 40 years, precast beams = we are obliged to use a large n° of beams TO REDUCE THE WEIGHT of EACH BEAM TO RENDER EASY THE TRANSPORT AND BUILDING.

Generally for cast in situ the interaxis is 5÷10m (not less 5m) with precast beam is i = 2.5/7m (more generally 2.5/3m → using the thickness of slab). → same core

TRANSVERSE BEAMS are NECESSARY AT LEAST ON THE BEARINGS because when I HAVE A TORQUE MOMENT IN LONGITUDINAL BEAMS, when I ARRIVE with M_T ON THE BEARINGS AND UNDER THE BEAM THERE IS GENERALLY JUST 1 BEARING IT IS IMPOSSIBLE TO CLOSE THE M_T I NEED 2 BEARINGS → REACTIONS IN BEARINGS GIVES A RESISTING COUPLE = RESISTING BENDING MOMENT.

Usually we have only 1 BEARING under each beam, it's necessary to have a transverse beam for which the M_T arriving by longitudinal beam becomes a bending moment in the transverse beam

In the past transverse beams were used along the span not only in correspondence of bearings. You can omit these beams if you use an enought thick slab (26/28 cm). We can omit transverse beams but we have to see effect of slab and transverse beams. When we decide to use T. beams because some authorities require them → ex railway companies → consider that one transverse beam in mid span has 1/2 the same effect of 2 transverse beams put at 1/3 of the span.

It's better to introduce 1 T. beam = because construction of T. beam is complicated, they should be built on site and generally is dangerous.

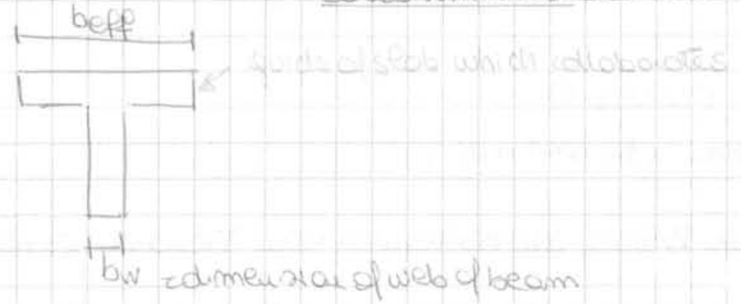
After mounting the l. beam, no difficult to build the t. beam on bearing because you're on the abutments or on the pier = workers on safe. BUT if you have to build along the span 1 or more t. beam it's difficult because there is no support for workers → create platform → difficult / expensive.

What about thickness of web? THICKNESS of WEB NECESSARY TO CARRY THE SHEAR → then higher is n° of beams → smaller is the thickness of webs → large n° of beams → a lot

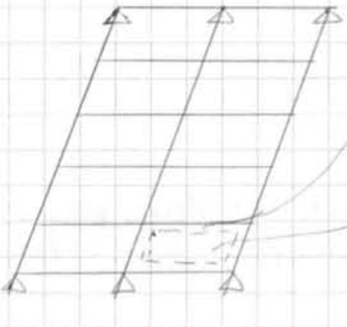
Different methods: aim of all methods is to arrive to the definition of internal actions in longitudinal transverse beams and in the slabs so that we are able to perform the design of l. beams and slabs.

- Courbon → advantage, very easy to apply (by hand calculation) → means that in the stage urban planimeter design used
  Also used for construction design (shop drawings).
  Introduce some semplification = pay attention to remain inside the field of application of the method
  This is a general engineering approach → in design you have semplified methods but you may apply only in within application fields
- Massonnet method = more complex and it takes into account in better way the actual behaviour of slab. (beginning of 60) and established an analogy between ortotropy of material and on geometry of slab. It was very used in the past = no more now because when we have to apply a refined method we use FEM.
- Method suggested for construction design which is in use of finite elements in ≠ ways girder structure composed only by linear elements or 2D/3D elements = level of complexity is increasing and also what it's very difficult is post processing =

Why we need such system to analyse C.B because this particular kind of structure is a space structure (l/t beams and slabs) receives vertical ~~forces~~ and horizontal forces in all directions → space SYSTEM → we can reduce to plane system in which elements are GIRDERS (l. beams and t. beams)
each are composed by web and a region on web which collaborates with web and the other elements of our slabs.



bw = dimension of web of beam



Continuous structures with l and t. beams with T section
We have substituted the continuous slab with a girder of beams orthogonal or not in which the region in which slab collaborates are
No slab here
~~TRANSVERSE~~
When we have to design for slabs take into account of continuous.

INTERNAL ACTIONS defined as COMBINATION of EXTERNAL ACTIONS → select the load case which may be unfavorable for structural region that we are considering.
Definition of loads is important because live loads are moving.
This kind of approach by means of Courbon method implies a definition and evaluation of transversal distribution of actions that is if I have P=1 in this position what % of this load is carried by this beam and by 2 and 3°? Because there is the participation of all the structure carrying the applied loads. Of course this load isn't carried only by the 3° beam.
Evaluation of this effect of collaboration of all deck in transferring loads to beams implies the evaluation of TRANSVERSAL DISTRIBUTION COEFFICIENT.

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d) TRANSVERSE BEAMS WITH FINITE FLEXURAL RIGIDITY AND BEAMS WITH ∞ TORSIONAL RIGIDITY

in carrying the action l.b has ∞ torsional rigidity = can't rotate only translate vertically. T.b can be bend because flexural rigidity ≠ 0 but finite → deformation ≠ settlements in ≠ beams and t. beams is obliged to have contraflexure points when there is a change in sign of moment

$\vartheta_P = \infty$
$\rho_E \neq 0$

e) TRANSVERSE BEAMS AND BEAMS WITH FINITE FLEXURAL AND TORSIONAL RIGIDITY

Rotation of beams and bending of t. beams the shape is that of reality. All l. b. can rotate and t.b. are bent.

$\vartheta_P \neq 0$
$\rho_E \neq 0$

f) TRANVERSE BEAMS WITH ∞ FLEXURALE RIGIDITY AND BEAMS WITH ∞ TORSIONAL RIGIDITY

Both beams with ∞ torsional rigidity → l b can't rotate
+ t b bent

→ when I apply P=1 only movement is translation in vertical plane and load is divided in 3 parts (uniform distribution of load) on the 3 beams

$\vartheta_P = \infty$
$\rho_E = \infty$

MAIN PARAMETERS THAT INFLUENCES THE GIRDER BEHAVIOR ARE

- FLEXURAL RIGIDITY of T.B and L.B
- TORSIONAL RIGIDITY of L.B

If I use l. b with high section the torsional rigidity is small (= 0)
" " b shaped beams = in which when I cast the slab I have a box section = the torsional rigidity isn't ∅ so it must be taken into account = I can't use some simplified method.

MAIN ANALYSES PROCEDURE

- Finite elements (combination of slab and beams)
- Courbon method valid for case ∞ rigidity of t.b, ∅ torsional rigidity of l b. { that means deck remains plane after application of load.
- Orthotropic slab analyses. Massonet.
  ↓
  2 preferential direction for orthotropical and general for the bearing capacity
- Beam girders → equivalent plane girder in which we consider the presence of longitudinal and transverse beams which collaborate with strip of deck.

Important used Massonnet = if you must repair a bridge of 60/70 I find the calculation with this method.

Method more used are Courbon → analytical calculation
conceptual design and preliminary design.

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Beam girder (plane structure of l/t beams) which requires use of computers for detailed design and shop drawings
drawings used for construction

$$\Sigma P_i d_i = 1e = \Sigma(K_i \delta + K_i \varphi d_i) d_i = 1e$$

$$\underset{\text{constant}}{\delta} \Sigma K_i d_i + \Sigma K_i \underset{\text{constant}}{\varphi} d_i^2 = e$$

$$\varphi = \frac{e}{\Sigma K_i d_i^2}$$

With 2 equil. condition I derive the new position of deck function of 2 parameters (δ, φ) then I know configuration after deformation.

Finally

L.B. ≠ Flexural rigidity

$$P_i = P_{ie} = \frac{K_i}{\Sigma K_i} \pm \frac{K_i e d_i}{\Sigma K_i d_i^2}$$

% of load in b. i = % of P=1 applied with eccentricity e on deck acting on b. i

+ e and d_i with same sign
− e " " " different "

if beams are same, and they are simply supported → K_i = constant

$$P_{ie} = \frac{K}{nK} \pm \frac{K e d_i}{K \Sigma d_i^2} = \frac{1}{n} + \frac{e d_i}{\Sigma d_i^2}$$ COURBON EXPRESSION (end of '70)

If I consider a generic beam (d_i = const) → the influence line $P_{i,e}$ can be drawn for variable i → what happens in beam i when eccentricity of load is variable.

What happens for load on beam i for variable position of acting load P=1

Beam 1/4

eccentricity / dist. same → G in middle distance of b1 to G = 1.5

$$P_{1,e} = \frac{1}{4} + \frac{1.5e}{2(0.5^2 + 1.5^2)} = 0.25 + 0.3e$$

$P_{1,1} = 0.25 + 0.3 \cdot 1.5 = 0.7$ → load on b. 1 70% of load is carried by beam n° 1

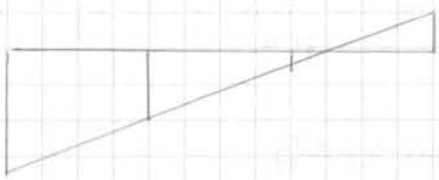
$P_{1,2} = 0.25 + 0.3 \cdot 0.5 = 0.4$ → load on b 2 40% of load on b. 1

% of action of load on beam 1 when load is in 1,2,3,4

≠ sign for eccentricity

$P_{1,3} = 0.25 - 0.3 \cdot 0.5 = 0.1$

$P_{1,4} = 0.25 - 0.3 \cdot 1.5 = -0.2$ → 20% of load applied upwards = downloaded



$$\sum_{i=1}^{4} P_{1i} = 1$$

I can apply Betty Maxwell Theorems → different interpretation to this approach = from the moment we evaluate the % of loads in beam n° 1 for 4 position = coefficient of transversal distribution of load.

According to B/M. T. I can say this one may be interpreted as transversal deformation 15

01/10/'08 ... GIRDER BRIDGES.

VINCOLO ELASTICO

Pg. 21 Theory of G.B is based on simple assumption that considers DECK TRANSVERSALLY RIGID AND BEAMS ARE CONSIDERED LIKE SPRINGS on WHICH THE DECK IS SUPPORTED.
Good hp: for deck which is narrow and long, but bad if deck is short.

For G.B. is a good way for doing predimension: no one uses Courbon Methods in design of a bridge → we have computers, aided programs → so we design the girder as a girder of beams with FEM beams. To choose dim beams or n° of prestressing tendons that I want to place inside the structure → before doing FEM in which we have to create n°, you have already known the section (programs enters data for cross section, number of tendons...) For predimensioning COURBON IS GOOD for GIRDER BRIDGES.
If I don't have experience I calculate M and internal actions in 2/3 points of bridge, choose dim of beams, n° of tendons and enter Courbon.

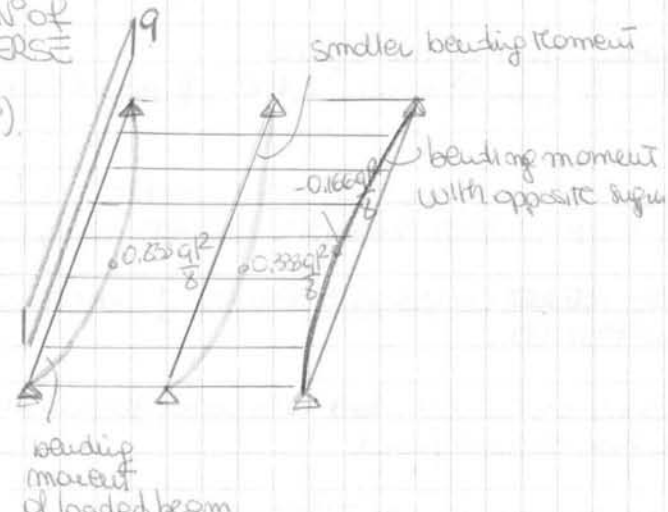
With simple equations of equilibrium to rotation and translation = calculate the F inside the beams with which I have a distribution of F when lateral beam is loaded and another when the inner beam is loaded

The METHOD WORKS FINE IF THE DECK IS TRANSVERSALLY RIGID → THICK SLAB, SMALL DISTANCE BETWEEN LONGITUDINAL BEAMS → SHORT SPAN so the SAME THICKNESS BECOMES STIFFER.
OR I HAVE A HIGH N° OF TRANSVERSE BEAMS ALONG BRIDGE.
In reality I never have an high n° of transverse beams along the bridge because trans. beams are discontinuity points → expensive.
Generally I have precast beams and trans. beams are cast in situ and prestressed in situ and so you try to have a small n° of trans. beams because they take a long time to be done = not economical.
Generally for G.B (span 30m) → 3 transverse beams → one on the first line of bearings
one in midspan
one at last.

So STIFFNESS IS ONLY A MATTER of DISTANCE of BEAMS
& THICKNESS of SLAB
Sometimes we use D SHAPED BEAMS AND BEAMS ARE TOUCHING → SMALL DISTANCE BETWEEN BEAMS.

COURBON WORKING FINE IN CASE of RIGID TRANSVERSE CONNECTION BETWEEN THE BEAMS so we have.

② HIGH N° of TRASVERSE BEAM (∞ N°).



These coefficients are taken from the table

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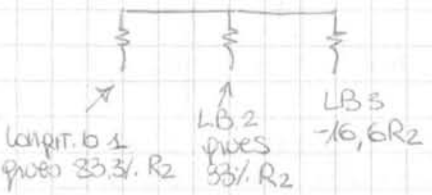
For having an idea of internal actions of Trans. beams we have to use E.M.

If n° of transv beam ≥ 3 difference with previous method is negligible.

HOW TO CALCULATE INTERNAL ACTIONS IN TRANSVERSE BEAMS.

For the load coming from the two reactions → only these 2 loads are giving rise to action on transverse beam.
I know p between the longitudinal beams → I know the repartition effect so I know and I use the scheme of beams on springs. I know reactions on springs → are the repartition coefficients.

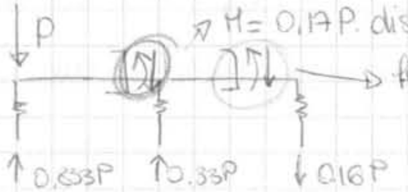


For a Force put on beam I know p and I superpose P to reaction (P can be directly on one beam or whatever position) –

Known F, Reaction = calculate actions M, V inside beam because equilibrium to vertical translation and rotation is already given when I calculate p.

From static point of view presence of P and reactions are already in equilibrium = only calculate internal actions → so for designer this beam isn't load by P and bearings but by P and other forces that I already know → calculate M, V in every point.

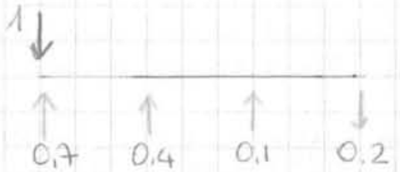
If I move P along transv. beam I calculate influence lines of M and V in transv. beam for ≠ position of P.



→ for calculate M I take into account 0,83P 0,33P with ≠ lever arm

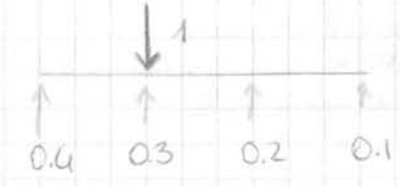
Easy to calculate M and V

If I MOVE P → ≠ REACTIONS → Calculate M and V in every section.
At end I have M and V for ≠ position of P. → influence lines of M and V.



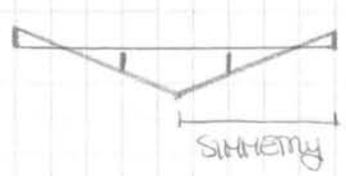
Midspan

$M_S = (0.7 - 1)\,1.5 + 0.4 \cdot 0.5 = -0.25$



$M_S = (0.4) \cdot 1.5 + (0.3 - 1) \cdot 0.5 = 0.25$

M in midspan



Variation of M in midspan of transverse beam = when the force is moving.

Influence line of shear in transverse beam → right side of beam 2

COMPUTER AIDED DESIGN of GIRDER BRIDGES

Pay attention in schematization of structure in a mathematical model → consider effects generated of stiffness, definition of girder mesh.

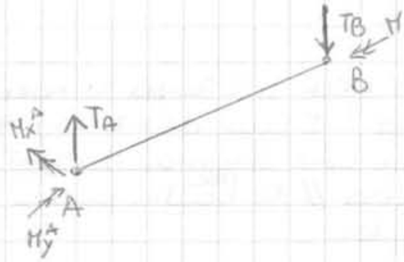
Method used: displacement formulation.
Consider a simple girder bridge of 4 longitudinal beams and 4 transv. beams → UNKNOWN THE NODES DISPLACEMENTS. AND IN GENERAL EACH NODE HAS 6 COMPONENTS of DISPLACEMENT but with simplified structure we consider just ~~...~~ PLAIN GIRDER with 3 UNKNOWNS
→ vertical displacement along z
rotation around x
rotation around y.

ex = 16 nodes

? = 16x3 - 8 = 40 can't solve by hand this case
↓
vertical disp. of points where there are bearings (z = ∅ for definition).

- SINGLE BEAM

A and B extremities → relation node displacement / internal actions acting on nodes may be expressed by S_i (vector of internal action on nodes) = $[K_i]\{\delta_i\}$
↓ stiffness matrix of elem.
↘ vector of displacement in nodes



We consider 3 components of displacements → the associated internal actions on the corresponding are in terms of displacements

Node A: vertical displacement = Shear TA in vertical plane
rotation around x = Bending moment M_{xA}
" " y = Torque " around axis of beam.

The same for node B

$$\{S_i\} = \begin{Bmatrix} T_A \\ M_x^A \\ M_T^A \\ T_B \\ M_x^B \\ M_T^B \end{Bmatrix} \qquad \{\delta_i\} = \begin{Bmatrix} z^A \\ \theta_x^A \\ \theta_y^A \\ z^B \\ \theta_x^B \\ \theta_y^B \end{Bmatrix}$$

$[K_i]$ 6x6 elements. Contain flexural and torsional contributes to rigidity of element.
Att! Reference to rigidity above all for concrete struct. take into account the cracked state → if I intend to analyse a girder bridge in R.C. only not P.C. also in SLS → deck is cracked, and long. and transv. beam are cracked
And then when I define rigidity take into account that cracking in bending reduce 50/60% flexural rigidity but torsional 3/4 times.
Att! if I proceed analysis from SLS to SLU (surely cracked also in torsion) → I have to introduce variation of torsional rigidity → imply ↑ of M over value expected and reduction of M_T more than we expect.
For classical analysis for SLS for P.C. girder bridges no problem → they have elastic rigidity.

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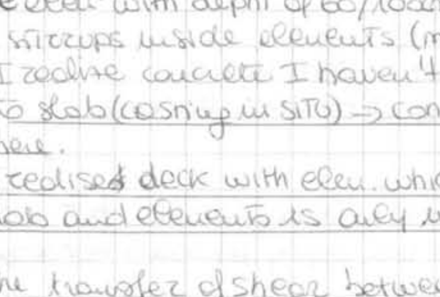
The solution is to have a global analysis with a limited n° of elements and then have a partial model which is more refined to extremities of which I apply internal actions coming from further analysis.
~~Two~~ step. to follow the analysis → general one → rough model
→ refined model → describes in detail only one region of slab.
When I consider the model ~~as~~ a part of global structure → apply to border of model some imposed deformations ~~deriv~~ which derive from global model
those " " , into 2 directions on effects of overall structure on this region.

- One typology for small bridges → precast elements connected by longitudinal hinges.
→ elements with no ordinary reinforcements but they have holes built by extrusion, shape so that we can realize by small casting longit. hinge between elements.
SHEAR KEYS

Tested for ~~usage~~ then for bridge deck generally used as bearings (thickness till 40cm);
~~Some~~ elem. produced with depht of 60/100cm → technological system which allows for introduction of stirrups inside elements (machinery moves along direction of casting and when I realize concrete I haven't reinforc. outside ∫ → probl. connection of those elem. to slab (casting in situ) → connection realised ~~putting~~ this joint putting reinforcement here.
I realised deck with elem. which have 2/3 holes inside but with stirrups and connection slab and elements is only in correspondance of shear keys (longitudinally).

The transfer of shear between elements it's possible by shear region → by friction.
Test shear for span ~~of~~ 20m also in continuity (continuity reinforc. in continuity region) and test for fatigue → resistance is based on shear friction not better for fatigue → reproducing the ↑ in stress variation (f = 1Hz ↑Δσ → reproducing effect of highway traffic = 100 mln of fatigue load model in 100 years) → good behaviour.
BUT PARTICULAR CARE ∅ THAT IS SHEAR INSIDE ELEMENTS AND SHEAR CONNECTORS.

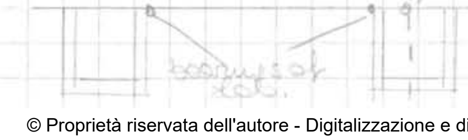
Useful for span of 6/8m → not this connection → only P.C.
Mutual connection is given by shear key (longit.) → I have to build model with longit. beams corresponding to element. Between 2 adjacent beams there is a longit. hinge so I introduce in transv. beam (dead axes). In the midspan the hinges.



Deck with: long. beams corresponding to actual elements
Transv. " on bearings (to evaluate correctly the internal actions in transverse direction).
But because it's unable to support M in → direction because I have just shear connection → I have to put one hinge in middle of transv. beams.
So → No M in transv. " but only shear hinges.

- Classical girder bridge: longit. beams
transv. "
cast in situ deck connected rigidly due to reinforc. to longit. / transv. beams → easy case because mathemat. model describes exactly this structure.

- V/ BOX BEAMS PRECAST and PRESTRESSED ARE OFTEN USED. Problem: how can I describe the model?



If I put long. b. in the middle (in gravity center) → mathmat. model doesn't reproduce very well the behaviour because increases the span of structures

(19)

Type of mesh → think about the DIRECTION of FORCES AND INTRODUCE ELEMENTS IN THIS DIRECTION.

Mesh for $\alpha \le 30°$ long. beams connecting bearings // α
transv beams // border.

Mesh for $\alpha > 30°$ without edge beams: long beams connecting the bearings
Trans // ⊥ to skew edge → better description of transverse M and M_T.

Mesh for $\alpha > 30$ with edge beams → maintain ⊥ of long. beams to borders, transv beams horizontal because edge beams describe its internal action (high M_T and M)

Mesh oriented along main direction the F are transmitted.
Structure receives loads in whatever position and then loads are trasferred → understand the direction of F.
↓
elements in this direction = good mathematical model → acceptable internal actions = economical solution
Wrong mesh → I can obtain results to take bearing capacity → transfer load but I oblige load to transfer in not natural way = overestimate the internal action + large cracks in serviceability conditions because I oblige F not having a natural direction to bearings.

- Curved bridge: rotation M and M_T → what is bending for one order ~~of reinforcements~~ is torsion for other one.
  Case of ramps → introduce radial beams (virtual) → radial + circonfer. beams = model. Similar to a sector of a plate following the principle direction of F.

- Skew bridges = high M_T → if there is the slab → no M_T. because slab high resistance to M_T → if I have a bridge composed by girder bridges → high M_T means high TANGENTIAL STRESSES INSIDE BEAMS →
  If I INCREASE DIM of BEAMS → ↑ TORSIONAL RIGIDITY → ↑M_T → become competition ↑ of rigidity and ↑ of internal actions → reduce internal actions by reducing torsional rigidity.

ex skew bridge section AA → displacements  due to ≠ of displacem.
I have a high of M_T to requilibrate
Also for BB high M_T → reduce torsional rigidity = no high M_T inside beam.

Box section on pier. Because longitudinally span wasn't the same → * due to variation of span → in presence of sismic action for eccentricity of box section → high M_T = implies uplift of bearing *.
Designer changes rigidity of box section = but it has very high rigidity and propose a special bearing able to transmit vertical reaction in tension = expensive.
Solution: take away 2 bearings* and put only one △ → I can't bear torsion M_T closed on other spans or abutments (here enough vertical reaction).

(20)

↓ INTERNAL ACTIONS ↓ RIGIDITY (FORCE GO IN DIRECTION of HIGH RIGIDITY)

SINGLE BEAM DECK

Used for example for a lane entering the highway (connection for highway).

Web can carry shear and Torsion (coming from live load).
What happens when we arrive at abutments or on pier?
M_T should be equilibrated by the 2 bearings → vert. reactions due to vertical shear, M → and M_T is a disequilibrium. → To equilibrate M_T
To have internal lever arm (small) → Force high bigger than the vertical ones on bearings → in this case, elastic one, I can't put one bearing I have to close of the end M_T.
Solution: INTRODUCE RIGID TRANSVERSE LOCAL BEAM → INCREASE WEB WITH IS WIDTH.
→ bearings are far = increased span of bearings → reduce the ammount of F.
Big beam → Transv. beam is designed for bending → receives 2 concentrated forces ~~due to~~ to Fule

On pier → not introduce trans beam crossing the pier (for abutment not a problem) → not ↑ width of pier → introduce artificially a preloading on bearings → when I have disequil. due to torsion I have the preloading that compensates → very easy.
Not so easy if there is possibility of longitud. movement ~~of deck~~ between deck and pier = I have to apply prestressing but I have to permit displacements → tendons should be very long free in lenght → so that movement of bridge implies very small ΔM in tendons.
Introduce some reservations in the pier = deck can move

ex → fixed cone inside concrete → large radius of curvature → curvature small → increase stresses inside tendons.

Pier with fixed beam I can't interrupt pier → otherwise I have tension on small region of pier = follow in tension → prolong the bars of tendons = enough weight of pier to compensate this

Pier small = extreme part of pier is in tension coupled by tendons.

PRESTRESSED BEAMS

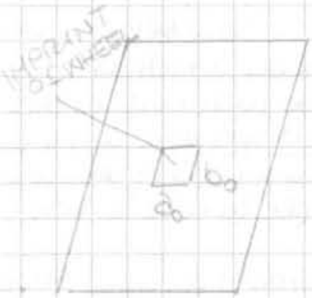
Prest. tendon = those which are in midspan/halfspan or upper region of beam when we are at a quater of ~~beam~~ span they are deviated because they propague parabolically but web ~~is~~ has a width that allows for the introduction of just one at beam. When I migrate tendons they have to enter the web but locally I have transv. M because push. isn't simmetrically respect vertical plane of beam. but it's a local effect.

I enter one then the other

In continuous beam, above all today, very long tendons couple of prestr. tendons are used → I introduce tendons in beam → finished beam → when I restart construction I use couplers → 2 enter coupler.
local weakness in structure so they are located in local region in which the stress level is lower that is in the counter flexure region where M is min. → continuous beam, are located at 0.2/0.25 span.
Start construction I span is 75% of current span + build I one + 25% of II then couple tendons. then other 75% + 25%.

ex 100 m of current span I is 75 m but I cast 75+25 Joint Tendons 75125...

When we have continuous beam we have alike hat ~~continuous~~ tendons in continuity region. Decide extension of hat
As a function of ~~M~~ opening of the ~~control~~ curved area of tendons you may generate M ≥ 0



Wheel is applied not directly to deck but to pavement → within thickness of pavement → dispersion of load and then imprint increases when I arrive on top of slab → ~~Refe~~ Generally I consider distribution of load to midspan of deck but experimental tests demonstrate it's safe because considering behaviour of slab we can also apply dispersion up to plane of reinforcement → generally it isn't accepted. Distribution of 45° in thickness of paving and midthickness of plate.

Finally imprint is

$$a = a_0 + 2S_1 + \cancel{2} S/2$$

effective dimension of wheel; 2 times thickness of paving; 2 times half the thickness of slab

$$b = b_0 + 2S_1 + S$$

Having distributed load → we have one or more distributed loads on central region. First approach → design as equivalent strip that is like this plate is very long → when ratio of this slab is more than 1/2 → the dimensional effects are very small. We can consider to have a very long beam. Practically the slab if

$ly/lx > 2 \rightarrow ly \simeq \infty$

distance of transv. beams; distance of longit beams

In this case for uniformly distributed loads → the deformed shape is cylinder → all sections are in same conditions.
~~Instead~~ If we have concentrated loads we have a surface with a double curvature. If we consider distributed loads → plane of traffic we add in addition to principle M responsible of this effect we have a SECONDARY MOMENT DERIVING FROM RESTRAINS EXERCED BY CONTINUITY OF MATERIAL TO DEFORMATION IS SAME DIRECTION → when I load one strip the adjacent strip exerces a restrain to loaded strip and this is the origin of secondary moment. We know

$$M_y = -\nu \frac{\partial^2 w}{\partial x^2} \left(\frac{E s^3}{12(1-\nu^2)}\right)$$ equivalent M. of inertia for slab

Poisson coefficient

II derivative of vertical displacement with respect of x 2 times = curvature in secondary direction

For beam the equivalent expression is $M_y = -\frac{\partial^2 w}{\partial x^2} \frac{E s^3}{12}$

If we assume $\nu = 0$ (fully cracked beam) → $M_y = 0$ fully cracked so I don't have the bearing capacity in secondary direction → equivalent of this slab is ~~that~~ a serial of strips which are cutted and completely independent.
It isn't true → if we put $\nu = 0.1$ → $M_y = 12\% M_x$
$\nu = 0.2$ → $M_y = 20\% M_x$ (usually)
high value for concrete → $M_y = 70\% M_x$ max value never reached because we have cracks in transv. direction
This happens for distributed loads.

NOT DESIGN SLAB ONLY FOR m_x → when we arrive NEAR JOINTS or at the END of the BRIDGE → the DISTRIBUTION WORKS ALSO IN TRANSVERSE DIRECTION

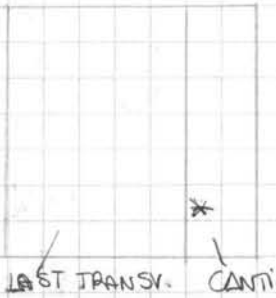
At extremities near joints or near abutments → the strip should be reinforced with twice times of reinforcements because M is twice. Not distribution on 2a but only on a.

Put load here

not distributed

When we design with such approach we have also deformed shape in longitudinal direction but having considered equivalent beam we have to design for only one direction. taking into account the deformed shape that means having a secondary M → introducing ordinary reinforcements otherwise we have very large cracks transversally to direction of deck.
Secondary M for long slab is 0.1/0.15 Mx → conservative width uses 25% Mx → design longitudinally reinforcement for slab for a M which is 25% of Principle M (transverse M).
At least 25% of transverse reinforcement in longitudinal direction.

DYNAMIC EFFECTS NEAR JOINTS (problem) → along slab and structure dynamic effects are included in load model of EC. But model doesn't cover what happens near joints



Cantilever arrives on joint
↓
longitudinal direction
If I consider only the cantilever → works well for dynamic effects introduced into model but I have a discontinuity → * localise effect. due to effect of joint I have some mm of ≠ between beam / cantilever → I have an impact of wheels against the slab and dynamic coefficients may be very large also of

LAST TRANSV. BEAM

CANTILEVER

order of 3 → I have to design longit. cantilever for strong dynamic effects generally $\phi = 3$ and taking into account the dispersion of this effect (strong M) → reinforcements necessary to border is extended to 1/3 of transverse distance of longitudinal beams or to a span of cantilever → ex 1m of cantilever → reinforc. extended at least for 1m or for 1/3 of transverse distance of beams if it's greater (ex. 2m of transv. beam → reinf. for 1m of cantilever).
3m is same

NOT LESS ANY CASE THAN 1/3

Att! Generally a lot of problems in girder bridges for joints → joints work badly → not able to avoid salt and water goes across joints → aggression of slab and piers. → slab cracked → flexural rigidity reduced → dynamic effect due to impact of wheels against slab is increased → forces greater and cracks greater and so on.
A lot of problems for changing joints (one in concrete) due to better design of slab.
Bridges of 60/70 → sometimes more 1m → system to reduce span of beams ~~absolutely~~ put minimum of joints because problem of durability and comfort.

For each span of slab do I need influence surface? NOT true = pratical use derives from invariability of influence S for slab that has same ratio of edge - ratio of edges for borders is the same the I.S have the same shape.

I know I.S → I can evaluate $\bar{l}_x$ $\bar{l}_y$ → I decide what is the ratio for ex 1cm is 2cm → I decide the imprint with this ratio and I design also the slab with this ratio when I have defined on the ratio $\bar{l}_x$ and $\bar{l}_y$. And actual dimension of borders l_x and l_y I can define SIZE RATIO = K

$$\frac{\bar{l}_x}{l_x} = \frac{\bar{l}_y}{l_y} = K$$ ex 1 cm of actual dimension → 1mm on drawing.

I define geometry, for load I use same ratio → if load is linear I reduce load of K (linear reduction) If it's applied on surface I reduce in both direction → the reduce of load is K^2

I read in this span the value of parameter G on I.S. if:

- concentrated load $G=\bar{G}$ G is exactly the value I find on IS
- linear load $G=K\bar{G}$
- surface load $G=K^2\bar{G}$

Geometrical ratio → reduce load by K or K^2 → evaluate internal actions by I.S I have multiplied by K/K^2

DRAWING PROCEDURES of INFLUENCE

Generally theoretical approach derives from derivative of displacement equations → Differential equation

$$\frac{\partial^4 w}{\partial x^4} + 2\frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^2} = \frac{-p}{B}$$ equation of slab

flexural capacity $= \frac{\quad}{(1-\nu^2)}$

if I solve the equation I know function w (Method of Pucher or Method of singularities)

$W = W_1 + W_2$

actual deform shape

By II/III derivative → I have M, V

General geometry of slab isn't regular → not easy to ∫ the equation only for regular condit. of restrain elements

w_1 = deformed shape near load of a ~~regular~~ circular plate loaded in the center ~~...~~ around load a circular plate (because I know condition col 57) → displacement is

local contribution $w = \frac{r^2}{8\pi B} \ln\left(\frac{r}{a}\right)$

distance

diameter

which are big because we have Max stress JUST at one level. Membrane stresses assume the more R → that is uniformly on the plate where tension is in two directions → and elastic field is reached

4) step: ↑load plastic hinges along longit. stiffners → plate shows plastic hinges in correspondence of bearings and membrane failure, due to deformation of plastic hinges M, is unable to ↑ further but ≠ stresses continues to ↑ up to the failure.

This system of stresses can't be simply summed → take into account → we evaluate with simplify approach → the sum is negligible
We may use the high bearing capacity of plate of ~~bearing capacity~~ for R but we must respect the SLS → and thus the limit in design doesn't derive by ULS but from deflection in SLS → otherwise isn't possible to main connected the paving.
In determine in design the thickness of plate we use the limit state of deformation to guarantee durability of pavement.
To avoid problem to paving

$$W = \frac{a}{300} \rightarrow \text{interaxis of ribs}$$
length

Design not for bearing capacity but for displacement.

BOBNOV APPROACH (experimental check by Kloppel)

$$W = \frac{a}{300} = \frac{1}{6}\,\frac{5}{384}\,\frac{pa^4}{EI} \rightarrow$$ behaviour of beam (1 direction)
→ plate here bearing capacity in 2 directions → we have membr. force which reacts to deformation.

for membrane action the value is reduced

that are of a plate being fully restrained

$I = t^3/12$

$t = 1.58\,a\,\sqrt[3]{\frac{p}{E}} \rightarrow t = 0.015\,a\,\sqrt[3]{p}$

$E = 210 \cdot 10^6\ kg/cm^2$

ex = pressure of tyres → 2 atm for truck →
" → 8 atm
$t = 0.015\,300\,\sqrt[3]{8} = 9\ mm$ → NO THICKNESS UNDER 10 mm.

$a = cm$
$p = kg/cm^2$

BEHAVIOUR of STIFFENED PLATE

After test of Kloppel and having understood what happens in practise = understand behaviour of stiffened plate.

- Most used procedure → stiffened plate is described as an orthotropic plate (2 prevailing directions of behaviour longit. for ribs, transverse for beams) → continuously supported by elastic restrains (similar to procedure for girder bridges) → Pelikan - Esslinger method.
- Consider deck like a girder with infinitesimal mesh (a lot of longitudinal beams and transverse beams and plate = deck) → Massonet (not possible for this geometry applying Courbon) _ Guyon Massonet method
- I.S. drawn for some regions
- NUMERICAL ALGORITHMS → for omogeneous/uniform thickness of slab → orthotropic slab.
" " that can be used for num. procedures (FEM) or variations method.

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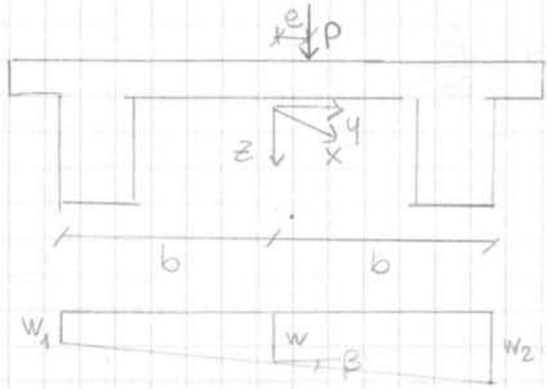
15/10/'09

TRANSVERSELY RIGID GIRDER BRIDGES

Analysed with theory of Courbon → but in reality this is an 2 beams that permitt same consideration.
In this kind of bridge the transverse section can be open or closed but unable to change the shape by effect of presence of relevant diaphragm system
Case for ex of bridges with 2 steel webs and diaphragm → is/2 rigid of girder. Or 2 concrete beams with significant n° of diaphragm.



For semplification → 2 beams (but we can have a n° of beams we want.) connected by slab.
On deck there is a load P (linear load on lenght of span) function of x (axis ⊥ to plane) applied with ecc. e
By effect of P because section is unable to modify the shape = ROTOTRASLATION of SECTION → def is the composition in every point of rotation + traslation.

$tg\,\beta = \beta$ (small angle)

$$\begin{cases} w_1 = w - \beta b \\ \\ w_2 = w + \beta b \end{cases}$$

w: mid displacement
b: distance of gravity and extreme beam

$\rightarrow w_1 + w_2 = w$

We know that by differential equation we can relate applied load with flexural rigidity of beam and displacement of beam

$$EI\frac{\partial^4 w_1}{\partial x^4} = EI\left(\frac{\partial^4 w_0}{\partial x^4} + b\frac{\partial^4\beta}{\partial x^4}\right) = -P_1$$

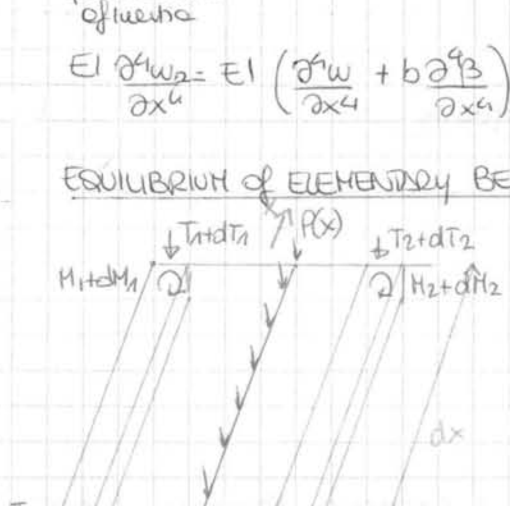
% of load acting on beam

EI: flexural iner of inertia

$$EI\frac{\partial^4 w_2}{\partial x^4} = EI\left(\frac{\partial^4 w}{\partial x^4} + b\frac{\partial^4\beta}{\partial x^4}\right) = -P_2$$

add → $$\boxed{2EI\frac{\partial^4 w}{\partial x^4} = -P_1 - P_2 = -P}$$ for equil. reasons.

EQUILIBRIUM of ELEMENTARY BEAM LENGHT (and also of deck)



We suppose deck with lenght of dx. If I isolate the element from deck → I torque moment and shear (on other face I have an increase).
P(x) has ecc.

EQUILIBRIUM AROUND LONGITUDINAL AXIS x

$$(T_1 - T_2)b + [-(T_1 + dT_1) + (T_2 + dT_2)]b - M_1 - M_2$$
$$+ M_1 + dM_1 + M_2 + dM_2 + \underbrace{m\,dx}_{p(x)e} = 0$$

$$-dT_1 b + dT_2 b + dM_1 + dM_2 + m\,dx = 0$$

$$\left(-\frac{dT_1}{dx} + \frac{dT_2}{dx}\right)b + \frac{dM_1}{dx} + \frac{dM_2}{dx} + m = 0$$

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$$2EI \frac{\partial^4 w}{\partial x^4} = -P \qquad 2EIb^2 \frac{\partial^4 \beta}{\partial x^4} - GI_t \frac{\partial^2 \beta}{\partial x^2} = m(x)$$

$$E \sum_i^{n} I_i \frac{\partial^4 w}{\partial x^4} = -P(x) \qquad EI_{yz} \frac{\partial^4 \beta}{\partial x^4} - GI_t \frac{\partial^2 \beta}{\partial x^2} = m(x)$$

$$\begin{cases} E \sum_i I_i \frac{\partial^4 w}{\partial x^4} = -p(x) \quad (*'') \\ EI_{yz} \frac{\partial^4 \beta}{\partial x^4} - GI_t \frac{\partial^2 \beta}{\partial x^2} = m(x) \quad (*''') \end{cases}$$

→ connected to flexural rigidity of beam and displacem. in vertical plane of G than of mid line of displac.
→ related to torsional rotation

2 differential equations govern the problem.

(*'') No ecc. so def is governed by flexural rigidity

(*''') 2 contribution to torsional rigidity. If I follow classical approach I have only the II term → in this case I have an addition

LIMIT CASE

- $J_t \simeq 0$ 2 steel beams → $J_{ps} \simeq 0$ How can torsion be carried? It's carried by differential bending in 2 beams able to carry the torsion.

$$EI_{yz} \frac{\partial^4 w}{\partial x^4} = m(x) \quad \text{similar to } (*'') \qquad \text{BI-MOMENT (Mom of I. of M of I)}$$

Torsion behavior governed by flexural rigidity of beams. Effect of M_T = unbalanced of beams one receives the shear upwards and the other downwards and they react flexurally.

- $I_{yz} \simeq 0$ if I_t very high so it's the only interesting → classical st of De S.V → case of BOX GIRDER → torsional contrib. high → omit the contribution of flexural behaviour.

For open section beams → $J_t \simeq 0$ effect of BI-MOMENT is PREVAILING
For box girder bridges GJ_t high → prevailing.

The diff* equation can be integrated (IV order, linear, constant coefficients) → I can consider deck with doublet beams → I substitute by semplification the M_T by 2F opposite sign (equal value) applied in correspondence of webs. (= couple) → $M = F \cdot$ distance of webs. (2b)
The beams deflect in the opposite position (direction)

$$M^* = -EI \frac{\partial^2 (w_2 - w)}{\partial x^2} = -EI \frac{\partial^2 (w - w_1)}{\partial x^2} = -EI \frac{\partial^2 \beta}{\partial x^2} b$$

downward displ. — displac. in beam 2

bimoment behaviour = II torsional behaviour related to bimom. = flexural secondary behaviour = equivalent M^* in beams

$$M^* = -EI \frac{\partial^2 \beta}{\partial x^2} b \rightarrow \frac{\partial^2 \beta}{\partial x^2} = -\frac{M^*}{bEI} \rightarrow \text{substitue in } EI_{yz} \frac{\partial^4 \beta}{\partial x^4} - GI_t \frac{\partial^2 \beta}{\partial x^2} = m(x)$$

$$2EIb^2 \left(-\frac{1}{bEI} \frac{\partial^2 M^*}{\partial x^2} \right) + GI_t \cdot \frac{M^*}{bEI} = m(x)$$

$$\boxed{-\frac{\partial^2 M^*}{\partial x^2} + \frac{GI_t}{2b^2 EI} M^* = \frac{m(x)}{2b}}$$

solution for this simple case.

What is the effect of bimoment? That I have a flexural stress in beam → beam is bent downwards → along the web I design diaphrams of plane rotated by 90°
I have tension in flocav → compression in slab and opposite for II beam.

CREEP EFFECTS

3 PRINCIPLE of LINEAR CREEP → PRINCIPLE of REINTRODUCTION of DELAYED RESTRAINTS.

Elastic, homogeneus, body rigidly restrained
I apply a load by effect of load → in restrains
I have reactions ($X_i(t_0)$)
time of application of load
After application of load → I introduce a further restrain X_{n+1} → I put in contact with beam not forced. How much is the reaction? $X_{n+1}(t_0)=0$
Because restrain applied after application of load.

But in time I have had in the point a further displacement due to creep → with restrain I impede the displacement and then originate a reaction → $X_{n+1}(t_0) \neq 0$ and ~~other~~ reactions ~~change of $\Delta X_i(t_0)$~~ assumes a value $\neq \phi$ in time

What is value of reaction at ∞ time? I introduce the restrain before application of load so other reactions are $\neq$ of $\Delta x_i(t_0)$ because I have a further restrain

variation of $\Delta x_i(t_0)$
And in $n+1$ → $X_{n+1}(t_0) \neq 0$.
$\neq$ condition → come to previous case: in point in which I have applied a further restrain I put a imposed deformation → settlement corresponding to value of displacement in absence of the restrain $n+1$ → so reaction becomes again ϕ

$t=0$ $\quad n+1 \Rightarrow$ $\quad X_{n+1}(t_0) - X_{n+1}(t_0) = 0$

introduction of restrain — dependent by F
removal of settlem. of restrain — dependent by imposed def.

ORIGINAL CONDITION.

$$i \rightarrow \underbrace{X_i(t_0)}_{\text{for load}} + \underbrace{\Delta X_i(t_0)}_{\times\text{load}} - \underbrace{\Delta X_i(t_0)}_{\text{effect of displacement imposed}} = X_i(t_0)$$

What happens for evolution of time with creep?

A = derived from system of F and so for I principle can't change in time
B = " " imposed displacement by II " is variable in time with a relaxion to load
C/D = invariable in time
E = variable in time

Generic t

Relaxation function

$$X_{n+1}(t) = X_{n+1}(t_0) - X_{n+1}(t_0)\frac{R(t,t_0)}{E_c} = X_{n+1}(t_0)\left(1 - \frac{R(t,t_0)}{E_c}\right)$$

$$X_i(t) = X_i(t_0) + \Delta X_i(t_0) - \Delta X_i(t_0)\frac{R(t,t_0)}{E_c} = X_i(t_0) + \Delta X_i(t_0)\left(1 - \frac{R(t,t_0)}{E_c}\right)$$

If I consider $t_0 = 28$ ~~years~~ days $\quad t = \infty$ (ex 30 years) → $\frac{R(t,t_0)}{E_c} = 0.15 \div 0.30$

function of R of concrete, moisture and etc

$X_{n+1}(t) = (0{,}70 \div 0{,}85)\, X_{n+1}(t_0)$

IF I INTRODUCE IMMEDIATELY AFTER APPLICATION of LOAD a FURTHER RESTRAIN IN STRUCTURE IN MOMENT IN WHICH I INTRODUCE RESTRAIN → CONTACT WITH STRUCTURE REACTION IS ϕ BUT FOR LONG TIME REACTION IS 70/85% of REACTION I HAVE IN POINT IF RESTRAIN HAVE BEEN INTRODUCED BEFORE APPLICATION of LOAD.

Generalize 3° principle → introduction of one/more further restraints at time $t \neq t_0$ but all together at t → not general case → so follow step → introduction of whichever n° of further restraints each one applied at ≠ time.
Limitation → homogeneous structure = concrete it's the same = not true I build bridge progressively with ≠ casting also concrete is same at origin but cast in ≠ time ⇒ ≠ ages
We omitt with application of principle but we have to take into account that speedness of construction is high (every cast 1 week) = effect of ≠ age small compared to long time we consider.
Generally for c.a. = homogeneous → ordinary % of reinforcement doesn't give nonhomogeneous material. (2/3%)
↓
We apply the 4° principle also if we cast in sequence

% steel is large → not apply the 4° → step by step approach
If time of construction not large: EEAM → simplified method.

29/10/08 4° PRINCIPLE of VISCO ELASTICITY

III It's not enought for the actual procedure in construction of bridges. IV extends a little bit the 3°

It considers a construction process in which there is a modification of restraint condition in homogeneous structure from statical scheme (1) to (2)

In statical scheme 1 k RESTRAINTS
2 $m > k$
structure is able to change statical scheme - How can we change? By introduction of $m-k$ RESTRAINTS AT TIME $t_1 > t_0$
application of permanent loads.

Variation of statical scheme applied in a time $t_1 \neq t_0$ with ≠ restraints

$X_R(t)$ = reactions at time $t > t_1$ in the k existing restraints → it means r is variable between 1 and k and this the family of first restraints

$X_S(t)$ = reactions at time $t > t_1$ of $m-k$ additional restraints s is variable between $k+1$ and m → adding further restraints at time $t > t_1$ I expect to have reactions.

If r are elastic restraints → so $X_R^{el,1}$ = elastic reactions in scheme 1 in which we have only k restraints

ΔX_R^{el}, ΔX_S^{el} = correction to apply to elastic solution in static scheme (1) to respect the further additional geometrical conditions (= m-k) imposed by additional restraints supposed to be introduced before the introduction of loads (scheme 2).

As 3° principle → def Δx for reaction variation of elastic solution in case of an applied further restraint for application of permanent loads. This is the same but in III principle = 1 additional restraint, here m-k additional restraints introduced in t_1.

There are some REACTIONS DERIVED BY FORCES AND I PRINCIPLE of LINEAR VISCOELASTICITY
They stay CONSTANT IN TIME (only DEFORMATION INCREASES).
In PRESENCE of PERMANENT ACTIONS THE ELASTIC VALUES OF REACTIONS $X_R^{el,1}$ (evaluated on elastic scheme n° 1 and consequence of perm. loads) REMAIN CONSTANT AND THE DISPLACEMENTS RELATED TO THESE SYSTEM of FORCES (u_S^{el} = displacements in every point of structure evaluated on static scheme (1), taking into account the E at moment of evaluation, increase due to creep by means of non dimensional factor which is $E_c J(t,t_0)$

$$\xi(t,t_1,t_0) = \int_{t_1}^{t} R(t,\tau)\, dJ(\tau,t_0)$$

So

$$\Delta X_R(t) = \Delta X_R^{el}\, \xi(t,t_1,t_0)$$

$$\Delta X_S(t) = \Delta X_S^{el}\, \xi(t,t_1,t_0)$$

Now we consider probably what are the values of reactions applying PSE

in m reaction restraints applied before application of load

$X_R(t) = X_R^{el,1} + \xi(t,t_1,t_0)\, \Delta X_R^{el}$ TOTAL REACTION AT TIME t

$X_R^{el,1}$ → independent by creep because related to system of F

$\xi(t,t_1,t_0)\,\Delta X_R^{el}$ → variable in time

$X_S(t) = \xi(t,t_1,t_0)\, \Delta X_S^{el}$ (No elastic contribution initially)

Practically the function $\xi(t,t_1,t_0)$ is a mesure of the part of REACTION DUE TO CREEP IN EVOLUTION of STATIC SCHEME WHICH IS ABLE TO MESURE THE PORTION of REACTION DUE TO CREEP of THE DIFFERENCE BETWEEN THE REACTION PATH CORRESPONDING TO APPLICATION of LOAD IN SCHEME 2 AND THAT ONE CORRESPONDING TO APPLICATION of ALL RESTRAINTS IN THE STATICAL SCHEME n° 1.

$$0 < \xi < 1$$

$\xi = 0$ $t = t_1$ same time I apply m restraints no contribution of creep = $\int$ is $\emptyset$

$\xi = 1$ $t_1 = t_0^-$ when I apply all the restraints before the application of the external actions I apply m-k restraints in $t = t_0^-$ and permanent loads in $t = t_0^+$ I have no evolution of statical scheme = I apply all restraints before application of loads.

If I consider the case of III principle = I apply the II system of restraints JUST immediately after the application of the loads

$t_1 = t_0^+$ for reaction

$$\xi(t,t_0^+,t_0) = \int_{t_0}^{t} R(t,\tau)\, dJ(\tau,t_0) = 1 - \frac{R(t,t_0)}{E}$$

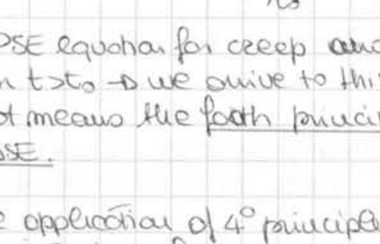
(generic) same value of III principle → if we go back and substitute →

In PSE equation for creep and shrinkage we introduce the reaction $\emptyset$ for $t<t_0$ and 1 for $t>t_0$ → we arrive to this equation.

That means the forth principle of linear visco elasticity has the III one as a PARTICULAR CASE.

The application of 4° principle implies we build the bridge practically at same time and then we introduce further restraints, corresponding to variation of static scheme, JUST in one time

ex. case of bridge with 2 hammers.



t_0 = time of application of loads (prestress + dead load)

I admit it's the same for two hammer (time of construction).

I wait same time and I apply the continuity. For this condition I can use the 4° principle

Generally we have several time of construction in which the external actions have variations due to introduction of further restraints →

Once I can describe the system: I pour at first place the segment one (I time) then I post-tension web on each side (II time) then I cast the concrete of bottom slab (III time), then I cast the concrete of top slab (IV time) → I can describe cantilevering on # edges → then prestressing (V time)....
We have a long process (long to be described) → all structure is assembled → analysis of internal actions → step by step what happens for each element, for each event ...

The 5° principle: (don't need to describe separately each event) and in application we perform some simplification = age of concreting is the same (also if we are ≠ ages during construction but if I have one year (time of construction) and I evaluate on 70 years the effect of creep = small error) and t_0 is the same (in fact it isn't because time of application of permanent loads ≠ time of construction)

Application of 5° principle is very fast.
Precision is high → error is within 10%. Compared to shrinkage and creep coefficient of variation which varies of 30% → 10% is nothing

$\varphi = 2{,}5$ min value = what we expect is $\varphi = 2{,}5 \pm 30\%$.

So it's not the case to use a very refined method to follow the construction if we have these coefficients so variable.

PRINCIPLE of REINTRODUCTION OF DELAYED RESTRAINTS.

- CONSTRUCTION PROCEDURES (influence)
- VERIFICATION IN LONGITUDINAL DIRECTION (imp!) but ~~also~~ for overall actions (internal actions).
- imp! DEFORMATION AND CAMBERING.

- Construction procedures.

Static scheme variation = only one or construction time = short we may use the 3° principle. (for M, T, N).

$$M_\infty = \underset{\substack{\text{evaluated on initial}\\\text{static scheme}}}{M_i} + \underbrace{\Delta M_i}_{\substack{\text{difference in reaction between}\\\text{initial/final static scheme}}}\left(1-\frac{R(\infty,t_0)}{E_c}\right) = M_f - \Delta M_i \frac{R(\infty,t_0)}{E_c} = M_f\left(1-\frac{R(\infty,t_0)}{E_c}\right) + M_i\left(\frac{R(\infty,t_0)}{E_c}\right)$$

LINEAR COMBINATION of M (or INTERNAL ACTIONS) EVALUATED ON INITIAL AND FINAL STATIC SCHEME

$t_1 \neq t_0$ → ~~apply~~ function ξ → 4° principle

for each segment $M_\infty = M_i + \xi(\infty, t_1, t_0)\,\Delta M_i$

$$M_\infty = M_f\,\xi(\infty,t_1,t_0) + M_i\left(1-\xi(\infty,t_1,t_0)\right)$$

We have a MIGRATION of INTERNAL ACTIONS (REDISTRIBUTION) = we have AN OVER RESISTANCE WITH RESPECT TO INTERNAL ACTIONS. When I build cantilever I have a lot of R because it's an open cantilever → when I introduce continuity tendons and cast keys migration of internal action from <0 to >0 M

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- Evaluate evolution of stresses due to dead weight, and then II portion of prestress lossing (for evolution of restraints).
- Evaluate of asymptotic value of effect of delayed restraint
- Evaluate prestressing losses (final scheme).
- Application of permanent actions (barriers, kerb...)
- Individuation of load cases due to variable actions (in position in which I introduce influence lines → when I have worst effect for position of load.

Using computer structure is described progressively → step by step verification of joints, weld parts...

Regularization of hammers, jacks. For precast elements / For cast in situ segments geometry is corrected step by step → I expect a set of geometry during construction → I measure what happens and considering the expectation we correct step by step the geometry of bearings.

- Deformation and cambering

Respect the design → long. shape and horizontal shape
Studied taken into account permanent action, not variable (further def).

- self weight, top prestress. → initial scheme
- Bottom prestressing → intermediate scheme variable
- Pavement, kerbs, barriers → final static scheme.

Variable actions (final static scheme) but they don't enter problem of cambering.

In general picture consider evolution of deformation → ε function of t → evolution due to permanent actions (is like to that for creep)

δ' = deformation which intervenes in initial static scheme $(t_0<t<t_1)$ during construct. of hammer
δ'' = deformation in final static scheme $(t_1<t<\infty)$

In general

initial static scheme (consider portion $\varphi(t_1,t_0)$ of creep)

final static scheme (consume remaining portion of creep)

$$\delta(t) = \delta'\varphi(t_1,t_0) + \delta''\varphi(t,t_1)$$

displacement expected in a point of section

$$\delta_\infty = \delta'\varphi(t_1,t_0) + \delta''\varphi(t_\infty,t_1)$$

$$\varphi(t_\infty,t_1) = \varphi(t_\infty,t_0) - \varphi(t_1,t_0)$$

$$\delta_\infty = \delta'\varphi(t_\infty,t_0) + (\delta''-\delta')\left[\varphi(t_\infty,t_0) - \varphi(t_1,t_0)\right]$$

displacement expected in each section → I have to correct the profile of construction to take into account I will have displacement in construction and during evolution of time

Cambering is evaluated as difference between expected profile at end construction (and at ∞ time) with respect to theoretical profile.

It's good to overestimate the cambering for covering possible mistakes = better to have $c>0$ cambering respect <0 (idea of unsafe bridge) — creep more than expected

An acceptable cambering is $\pm 1/2000$ of span $\pm$ displacements for live loads.

- HIGH TORSIONAL STRENGHT of section → TORSION IS BROUGHT BY BREDT MECHANISM AND HIGH RIGIDITY
- SUITABLE SOLUTIONS for CURVED BRIDGES → SUITABILITY FOR PRECASTING ON SITE AND FOR LONG DISTANCE
- ELEGANCE THAT REDUCES ENVIRONMENTAL IMPACT = Att! to impact → balance environment impact take into account not only CO_2 production but also the effect with enters in environmental evaluation including also the cost.

1.2 MOST COMMON CONSTRUCTION PROCEDURES

- PRECAST SEGMENTS WITH MATCHCAST JOINTS MOUNTED WITH LAUNCHING GIRDER (eventually with temporary stays) → one of most common

- PRECAST SEGMENTS MOUNTED of FULL SPAN → I have a girder spanning from pier to pier → the steel girder is able to support the weight of total span of beam → I put on place the segments → when I have assembled I introduce prestressing (external generally). And then I continue by prestressing with the previous spans (A lot of bridges in South America).

- CAST IN SITU CLASSICAL CANTILEVERS

- INCREMENTAL LAUNCHING I built some region of bridge near abutments then I add a STEEL NOSE and I push → when I push a I add a new segment

FORMWORKS

- Balanced cantilever construction.
  Two stations partially crane which support a suspended formwork → cast segments → prestress segments → move completely of lenght of segments → move the scaffolding plus the formwork
- Classical cantilever with auxiliary stays.
  Big spans to avoid to change section I use some temporary stays for construction of bridge
- Classical cantilever with launching girder. Typical launch. g. support the 2 formworks → There isn't the movable scaffolding connecting to formworks but the launch. girder and formwork is movable along launching girder →
- Launch of scaffolding
  launching of bridge on an auxiliary beam → I push along the auxiliary beam and the incremental launching with the steel nose

EXTIMATION

Idea in preliminary design of bridge cost

- CONCRETE NECESSARY FOR BOX SECTION MAY BE DEF AS AN EQUIVALENT THICKNESS OF SLAB. → so what is thickness of slab in which I pour all concrete.

$t_m = 0{,}35 + \frac{0{,}45\ell}{100}$ (ℓ → span in meter) unit of mesure $\frac{m^3}{m^2}$ ↗ of concrete ↘ of deck

- PRESTRESSING STEEL → function of span
  $A_p = 4{,}5 + 0{,}5\ell$ kg/m² → projection of segment in span.

If I use those values for variable depth

$l/d \leq 10 \div 15$
$d_s/d_m = 1,5 \div 3$

the slenderness ratio is

$l/d_s = 12 \div 24$

$l/d_m = 18 \div 72$

When I have VERY HIGH PIERS (TO AVOID TORSIONAL EFFECT) → I HAVE SPACE SO PUT NOT ONLY ONE COUPLE of BEARING BUT 2 ▷ I HAVE A SMALL SPAN BETWEEN THEM ▷ so in presence of 2 bearings we have a fully restraint for variable loads so M>0 is reduced in span but we have to pay att! to reactions = fully restraint = there is a M = disequilibrium of forces AVOID THE REACTIONS, UNIFORM DUE TO DEAD WEIGHT, HAVE DISEQUILIBRIUM due to VARIABLE LOADS GREATER THAN THE REACTIONS DUE TO LOADS BECAUSE IN THIS CASE I HAVE UPLIFT
TO AVOID THIS → CHECK INTERNAL SPAN BETWEEN BEARINGS (l_i) / CURRENT SPAN

$l_i/l \geq 1/20$

ex span = 100m → l_i = 5m between bearings (at least).

TRANSVERSE DESIGN

ONE CELL SECTION for ECONOMY of the FORMWORK

- ONE CELL when ratio $d/B \geq 1/5 \div 1/6$
- MULTI CELLS $d/B < 1/6$ (more of two cells)

When I couple two box girders take into account ▷ 1 bear over each box section ▷ torsion taken only by slab.
When arrive torsion the slab has an up! transverse M Bodsz = strong engagement of slab in transverse direction ▷ to design bear and distribute the eccentric load between the 2 box sections.

Once we use dispersion = put two bearings after each box section ▷ torsion is closed inside the central beam. Otherwise with one bearing we have to introduce a lot of transverse beams ▷ diaphragms also precast able to maintain shape of section of beam.

How design transverse section? Slide 2-2

Generally I need diaphragm with inspection hole, over the piers (only here) → sometimes diaphragms can be substituted by stiffening frames ▷ increase the depth of web of slab.
Att! substitution of bearings ▷ increase to distribute diaphragms or frames on a length → when I have to substitute I need some space for jacks ▷ not a problem of resistance ▷ displacement for shrinkage, creep and thermal variation but bearings should remained underweb in center of beam. If I put bearing on axis of beam → I have sliding of bridge the position of load move so I need along diaphragm because the load is taken by diaphragms
If I put bearings on axis of diaphragm ▷ diaphragm is small 50/60 cm
dim of bearings.
Bearings stay there only but when bridge moves eccentricity of load ...

a residual compression N (remains as longit. F in joints → able to press and to expell the epox resin

- At ULS for ex for M<0 → joints open → no continuity of slab but I design slab to be continuous then I need the shear transmitted between joints also when joints are open = I need shear keys (in top/bottom slab).

When joints open along web I need for equilibrium to transmitt shear → only if I have S.K. → transmission of shear is made by a compression region (by friction) and shear crosses the joints.

So they are usefull for verification at ULS

PRECAST SEGMENTAL CONSTRUCTION

- Assembling steps

I put first segment on pier I put on jacks to adjust at end of construction, I connect by means of tendons for equilibrium. Then I mount alternately the other segments by interposition of epox resin (generally pot life 1,5/2 h) put on segments arriving → I have 2 cleaned of the segment which has been mounted and put on the segment arriving
sometimes I prolonge pot life by addition of mixture to 3/9 hours. When I introduce final prestressing I have a II application of pressure on joint and II exposure of resin and better closing of joints.
In assembling I introduce temporary bars for assembling and put internal tendons → apply prestressing for int. tendons and remove bars for II assembling

Plano altimetric adjustment, by jacks, of cantilevers and casting the central key ($\ell \cong 40/80$ cm)

Apply after hardening of new casting = apply continuity tendons → put in tension from longer to shorter because = I applied prestress → generate on hyperstatic M>0 if I start from shorter in II joint I have M>0 so start from longer.

Grouting of tendons at the end of construction → difficult do it progressively with construction.

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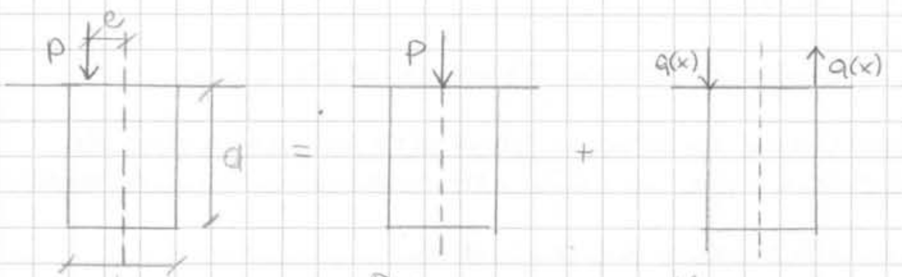
PRESTRESSED BOX GIRDER BRIDGES (2)

ANALYSIS IN TRANSVERSE DIRECTION.

Longit. analysis like a beam in 3D. Box section is a closed structure → closed diaphragm with 2 cantilevers. Check what happens when loads are applied centered or ecc. with respect of axis of bridge on the box = box should have bearing capacity in transv/longit direction

2.1 BREDT ANOMALY

We consider a box section with a load P applied with eccentricity e → P is able to produce transverse actions because I have M in slab and in other internal elements → It's able to produce also a M_T



P with ecc e

P along axis (not distorsion)

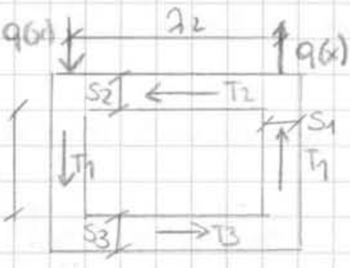
Mor. x ecc.
$M = q\,b$ → couple of 2 F acting at level of web
$\boxed{q = Pe/b}$

Closed section = I apply Bredt analysis.

Specif value of membrane stress is $t = \tau \cdot s = \frac{M}{2\lambda_1\lambda_2}$

tangential stress — thickness — Area included within the mean line of section

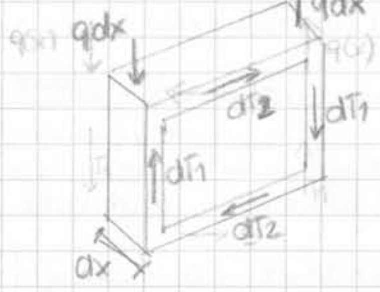
Resultant forces within web



$T_1 = t\lambda_1 = \frac{M_x}{2\lambda_2}$ (λ_1 = lenght)

$T_2 = T_3 = t\lambda_2 = \frac{M_x}{2\lambda_1} = T_1\frac{\lambda_2}{\lambda_1}$ (in slab)

Ignore cantilever because it doesn't participate
I consider an elementary segment of lenght dx and put in evidence the differential of the internal actions due to infinitesimal variation of the M_T

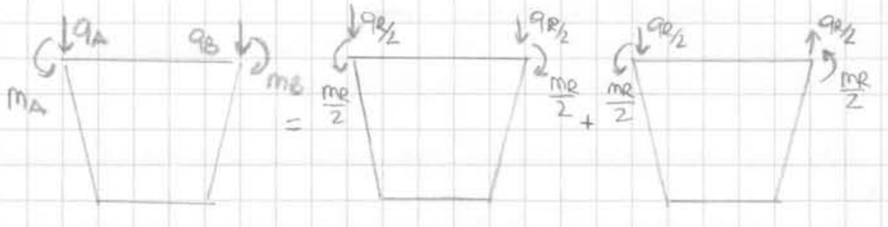


$dM_x = q\,dx\,\lambda_2$

$dT_1 = \frac{dM_x}{2\lambda_2} = \frac{q\,dx}{2}$

$dT_2 = \frac{dM_x}{2\lambda_1} = \frac{q\,dx\,\lambda_2}{2\lambda_1}$

I know F in each wall → consider the isolated element from rest of section → I see the external F $q\,dx/2$ aren't equilibrated by internal reactions $\frac{q\,dx}{2} = dT_1$ → so there are other forces inside able to restablish equilibrium → Bredt studies the global behaviour of section but when I analyse equil of each element of section we have to discover some other F which are inside the section



$q_R = q_A + q_B$ $\qquad$ $q_R = q_A - q_B$

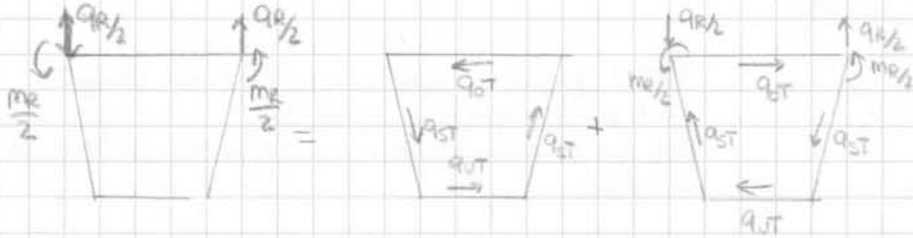
$m_R = m_A + m_B$ $\qquad$ $m_R = m_A - m_B$

SYMM CONDITION DOESN'T GIVE DISTORSION of ACTION, I HAVE OF COURSE INTERNAL ACTIONS

We consider a segment with unitary lenght with symm. loads and in equilibrium thanks to the difference of the shears in the webs on the segment faces (like segment supported along webs). Then we determine the TRANSVERSAL ACTIONS DUE TO THIS CONDITIONS = NO DISTORS. BECAUSE SYMMETRIC.

SUPERPOSING THESE STRESSES DUE TO SYMMETRIC ACTIONS → WE OBTAIN THE INTERNAL ACTIONS DUE TO SYMM. PART of LOAD

REMAIN ANTIMETRICAL PART THAT CAN BE DIVIDED IN
- A PURE TORSIONAL PART (q_{ot} q_{st} → Bredt part)
- A SYSTEM OF AUTOEQUILIBRATED ACTIONS → OPPOSITE of TANGENTIAL STRESSES + NODAL FORCES



For torsional part (Bredt) → consider already taught analysis → I have M_T → I design for this M_T, for shear we have an addition of torsion and so on

Condition ② for a lenght dx it results

$$dM_T = \frac{q_R}{2} b_o + \underbrace{m_r}_{2m_r/2}$$

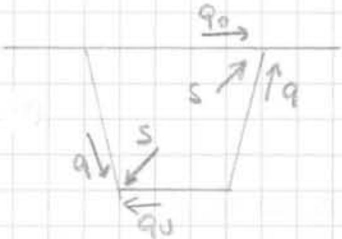
$q_{oT} = \frac{dM_T}{2A_K} b_o$ $\qquad$ $q_{ST} = \frac{dM_T}{2A_K} b_s$ $\qquad$ $q_{uT} = \frac{dM_T}{2A_K} b_u$ $\qquad$ $A_K = \frac{b_u + b_o}{2} d$

A_K → Area enclosed by mean line of section

I CONSIDER THE SEGMENT → I HAVE KNOWN FORCES → I know distrib of loads produce a distorsion of section → diaphragms inside section with high rigidity and evaluate F in diaphragms → they provoke variation of section because so rigid → To evaluate F due to this load condition and apply those F changed in sign to section without diaphragms.

Is it logical applying those F only to segment of lenght dx? If loaded condition is along the span → that is this effect of distorsion is local which requires the analysis of full span = I have this box section along → I load more segments = If I separate those segments from structure I appreciate that due to diaphragm forces changed in sign = distorsion of section → it's possible that I have distorsion without call the reactions from other part of bridge.

I divide a spread F into components q_s, q_o, q_u. I evaluate components along ≠ walls



$$q_o = \frac{S b_o}{8} \qquad q_s = \frac{S b_s}{8} \qquad q_u = \frac{S b_o}{8}$$ projection.

The web has q_s that gives bending/V in web = this F is deformation in section = bending and V in transverse direction

I consider the components of S carried partially transversally partially longitudinally.

$S(x) = S^*(x) + \bar{S}(x)$

S^*: % of S carried longit; $\bar{S}$: % of S carried of transv. compensable of distortion of section

$q_s \propto$ to $S \Rightarrow q_s(x) = q_s^*(x) + \bar{q}_s(x)$

q_s: load on web; q_s^*: % carried longit i.e. bending by web; $\bar{q}_s$: carried transversally.

Web is connected by means of hinges (ideal) in corner = so I have V ~~from~~ exchanged with remaining part of section distributed along span → T_o' T_u' (of course I have V at extremities of section) I assumed beam is slender

I consider effect of condition loads and restraints → induce M → in web I have a linear diagram of stresses σ_{xo} at ~~bott~~ top σ_{xu} at bottom → effect of load is to have a BENDING INSIDE.

THE COMPATIBILITY OF DEFORM. ALONG IDEAL HINGES CONNECTING WEBS = ALL HAPPENS LIKE I HAVE AN IDEAL MOM. OF INERTIA > OF ITS MOM. OF INERTIA. BECAUSE THIS WEB CAN'T BEND AS I WAS INDIPEND. BECAUSE CONNECTED TO REMAIN PART OF FRAME
Then there is an ideal I > of its I evaluated as a function of geometrical parameters

- IDEAL NEUTRAL AXIS → function of b_s (evaluated with non dimensionals

By effect of q_s^* I have longit. stresses in all walls evaluated by means of I_{si} with formula of Navier

$$\sigma_{xo} = -\frac{M_s(x)}{I_{si}} y_o \qquad \sigma_{xu} = \frac{M_s(x)}{I_{si}} y_u$$

y_o: distance of

out of plane stresses.

Because I report all as a problem of bending → also def of beam along vertical = function of q_s^*

$$EI \frac{d^4 v_s}{dx^4} = -\frac{d^2 M_s}{dx^2} = q_s^*$$

~~So~~ I HAVE ~~FOR~~ A % OF DIAGRAM FORCE → A BENDING OF WALLS = EFFECT = LONGIT. STRESSES → DISPLACEMENTS. → THEY AREN'T INDIPEND. WORKS WITH I > OF ITS I because they appartiento?

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In / of 4° derivative equations = respect boundary conditions -> rigid diaphragm at end of pier = I can't change shape -> like I have rigid beams (in this section) in elastic fund.

What is the worst position of load in transv. direction?
Influence line of displac. web (vs) in figure pg 13-13 -> The max effect is reached when resultant of load is at distance from centroid of $0.4 b_0$ = to have max effect put R here.
Only for very rigid section -> ↑ $v(s)$ for putting load at extremities of section (cantilever) -> effect -> more M_T, more distorsion.
90% of cases load is very near web ($0.4 b_0$ distance from G) not internal -> investigate for load a set of positions = longit. positions and transverse posit. to investigate. When I change position the effect is to have same longitudinal effect -> load condit. ≠ -> difficult.

Generally in this case we use a couple of position (relevant) -> concentrated / distrib loads.
e.l ↑ of longit effect with respect of dead loads = 30%.
D.l ↑ " " 10/20%.
↓
centered = less ↑

15/11/08

TRANSVERSAL ACTIONS INDUCED ON BOX GIRDERS AT THE VARIABLE DEPTH.

What happens in compressive chord of box girder bridge with variable depth -> we have compressed chord and resultant of compression ... along the chord but chord is curved = chord curved and it's compressed = so there is a transverse action (R) -> so uniformed distributed q load along chord due to compression which is at ULS -> $q = \frac{N_u}{R}$ N compressive force
R -> radius of curvature
We have such transversal action, at first time we suppose -> uniform distributed load oppose to dead weight (opposite direction) & problem: distribution of internal actions isn't uniform due to shear lag but it's not uniform of width x mean webs = distribution assumes a actual distribution effect variable due to shear lag and concentration of stresses near webs.
That corresponds in design to have an equivalent reduced width of chord.
This problem regards SLS = at ULS when we reach classification of concrete in region = redistribution -> to arrive to failure all chord works at fcd. In SLS esp. concentration of stresses -> means + creep def = ↑ def due to plastic actions (span of bridge)

To evaluate effect consider equiv. width of chord $b_{ui} = \frac{b_u}{2} \gamma$ (γ < 1)
for each side of section = for each web

Diagram variation of γ function $\frac{1}{\gamma} = \frac{\sqrt{R t_u}}{1.31 b_u}$
evaluated elastically because problem in SLS.

$2 b_{ui} = \gamma b_u$ instead of b_u we consider for verification $2 b_{ui}$
collaborated with web
To evaluate max σ
$\sigma_{min} = (1-\beta)\sigma_{max}$ internal actions are uniform along b_{ui}
uniform β varies between 0,5 ÷ -1

Don't forget of this in SLS.

What happens due to this load in terms of internal action (remember it's opposite to dead weight). Hence diagr. of compressive chord in transverse direction due to upwards distributed loads. Reactions aren't those for unif. distributed load for unif rigidity element but we have $M \geq 0$ which have elastic value

$m_m = \frac{q b_0^2}{24} \alpha_m$ — equiv. coeff to σ; elastic value
$m_t = \frac{q b_0^2}{12} \alpha_t$
α_t: 0.5 ÷ 1 (v of M > 0)

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it's +/- uniform F at max capacity on bearing ex bearing for 100 or 1000 tons has a settlem. of 1mm for max capacity → we can calibrate the rigidity of equivalent spring → so also control bearing receives F when MT arrives.

Considering case of rigid diaphragm is acting

T1 = shear coming from longitudinal analysis
T2, T3 = " " " primary Torsion (with Bredt evaluation)
T4, T5 = shears due to folded plate effects

Design for this system of F.

In case of intermediate bearing what happens due to strong reactions? Haunch in box → very often we are obliged to put prestressing in transverse direction in lower slab = extreme sz.

Susa viaduct = system of bearings. cantilever construct with cast in situ → proposed precast. but piers constructed = Not possibility to change design of piers = No space to put bearings.

INTRODUCTION of PRESTRESSING

Problems of STRUT AND TIE. Very difficult error in evaluation of M, V ... due to use of program that moves variable load on span to have max section.
Very difficult having errors in evaluation of longit. M = Problem is design of bearings, D regions, Regions of introduction of F.
Often = mistakes = cracks (= Problem of durability) or failure if in this region we apply prestress. is very dangerous for workers because acting F applied during prestressing.
so failure can be during construction.

HORIZONTAL SECTION of web = Tendon anchored inside the box. There is an enlargement of thickness of web and there is the anchorages tendons entering inside web.

Parameters to define are imp.! otherwise we enter the F inside in not correct way.

Enlargement must have a lenght of 12.50a
a = dim of anchor plate = which contains ... in which tendons are anchored.
The region in which we draw curve tendon ≥ 50
Radius of curvature ≥ 250

We distinguish zone (1/2/3) when we can def D REGIONS where Bernoulli/Navier theory isn't applicable = discontinuity region →

① High F concentrated from external
③ Distribution of ... forces due to curvature of tendons → put in tension tendons they push against concrete

1) Dim 2a backwards position of anchorage plate
4a upwards

3) Dim 8a

We use SYSTEM of STRUT and TIE simplification of compress. tracks / tension in steel. (40)

Suspension can be realised by tendons, bars.

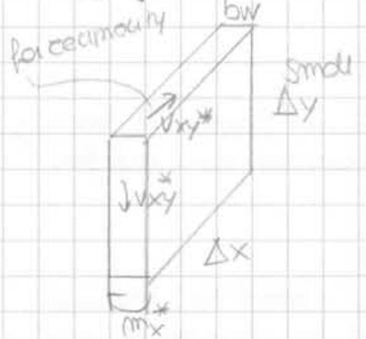
PRESTRESSED BOX GIRDER BRIDGES ③

INTERACTION BETWEEN LONGITUDINAL SHEAR AND TRANSVERSE BENDING IN THE DESIGN of WEB

Probl: interaction of internal action in web = due to shape of section = in web / longit. shear (from V and Mt) \ coupled with transv. M by actions of section of box in addition of folded plate.

For design web take into account there is V and transverse M. and transverse V. I can't consider 2 independent calculation = because 2 indip. calcul. = 2 ≠ reinf. → but concrete is the same of concrete is full exposed for V no resources for M.

Sandwich model
↓
Simplification → draw interaction diagrams



Element subjected to $V \to v_{xy}^*$, we consider small Δy (depth) not full depth.

Distributed actions → $m_x^* = \frac{M_x}{\Delta x}$
for unit length

$v_{xy}^* = \frac{V_{xy}}{\Delta y}$

Apply special type of sand. model considering only membrane forces omitting transverse shear is given only by box effect of section and by ~~...~~ box reaction.
In sand. there are 2 plates with ~~...~~ = ϕ thickness and shear is transferred by shear friction
Fin 2 panels n_y for m_x n_{xy1} effect of V
$-n_y$ n_{xy2}

$n_{xy1} + n_{xy2} = v_{xy}^*$ for equilibrium

thickness of panel

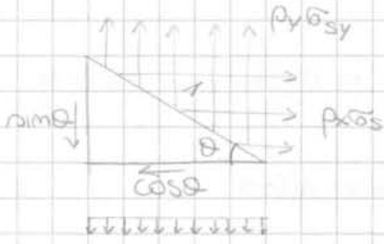
$n_{xy1} \frac{bw - z_1}{2} = n_{xy2} \frac{bw - z_2}{2}$ equilibrium rotation around axis of panel.

$n_y \underbrace{\left(bw - \frac{z_1 + z_2}{2}\right)}_{\text{lever arm}} = m_x^*$

If I know $z_1, z_2 \to n_y \; n_{xy1} \; n_{xy2}$

$n_y = \frac{m_x^*}{b_w - \frac{z_1+z_2}{2}}$ $\quad n_{xy1} = v_{xy}^* \frac{bw - z_2}{2bw - z_1 - z_2}$

In each layer system of F composed by σ_{y1} and τ_{xy1} (in each panel) σ_{y2} / τ_{xy2}



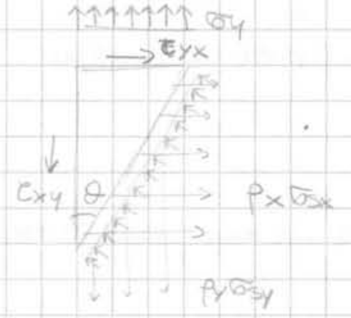
2 equations of equilibrium in vertical / normal direction.
CUT element in direct. of θ = in upper part reactions of steel in x/y direction. ($\rho_x \sigma_{sx}$ $\rho_y \sigma_{sy}$)
% of steel — stress in steel

We know from previous eq. we know σ_y τ_{xy}

? θ, σ_c

$\rightarrow \tau_{xy}\cos\theta - \rho_x \sigma_{sx} \sin\theta = 0$

$\uparrow \tau_{xy}\sin\theta + \sigma_y \cos\theta - \rho_y \sigma_{sy}\cos\theta = 0$



CUT along θ in ⊥ direction of I one so we obtain normal stresses in concrete along θ angle

$\rightarrow \tau_{xy}\sin\theta - \sigma_c \cos\theta + \rho_x \sigma_{sx}\cos\theta = 0$

$\uparrow \tau_{xy}\cos\theta - \sigma_y \sin\theta + \rho_y \sigma_{sy}\sin\theta - \sigma_c \sin\theta = 0$

If we manage the 4 equations

$\tau_{xy}\cos\theta = \rho_x \sigma_{sx}\sin\theta \Rightarrow \tau_{xy} = \rho_x \sigma_{sx}\,tg\,\theta$

$\tau_{xy} = (\rho_y \sigma_{sy} - \sigma_y)\cot g\,\theta$ $\qquad \tau_{xy}\cot g\theta + \sigma_y$

$\sigma_c \quad \tau_{xy} = \sigma_y \cot g\theta - \rho_y \sigma_{sy}\cot g\theta + \sigma_c \cot g\theta$

$\tau_{xy} = \cancel{\sigma_y \cot g\theta} - \tau_{xy}\cot g^2\theta - \cancel{\sigma_y \cot g\theta} + \sigma_c \cot g\theta$

$\tau_{xy}(1+\cot g^2\theta) = \sigma_c \frac{\cos\theta}{\sin\theta}$ $\qquad \tau_{xy}\left(\frac{1}{\sin^2\theta}\right) = \sigma_c \frac{\cos\theta}{\sin\theta}$

$\downarrow$

$1 + \frac{\cos^2\theta}{\sin^2\theta}$ $\qquad \boxed{\tau_{xy} = \sigma_c \cos\theta \sin\theta}$

We have limit conditions, Max value of ? $\sigma_c \rightarrow f_{cd2} \rightarrow$ cylindrical charact. design strength of concrete (found from prob of V) $\rightarrow$ compressive field but it's cracked along the same compress. Reduce quite a lot the compress. strength f_{cd} to take into account effect of stress. Pattern of crack is established consider f_{cd_2} as a value of bearing capacity for concrete

$\sigma_c \le f_{cd2}$ for concrete
$\sigma_{sx} \le f_{yd}$ for steel
$\sigma_{sy} \le f_{yd}$ "

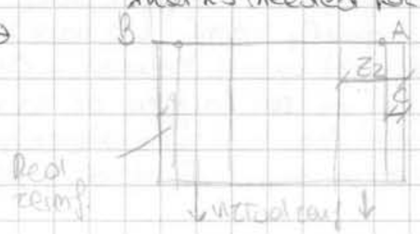
From these we obtain 3 inequalities with respect of τ_{xy}

$\tau_{xy} \le \rho_x f_{yd}\,tg\,\theta$

$\tau_{xy} \le (\rho_y f_{yd} - \sigma_y)\cot g\,\theta$

$\tau_{xy} \le f_{cd2}\sin\theta\cos\theta$

Problem of design of sandwich model. Reinforc. isn't perfectly in the axis of the 2 portions (z_1, z_2). We need some correction in calculation of reinforcem. that is needed for 2 ext layers because sometimes we choose to put reinf. at cover c. $c << z_1$ red reinforcement isn't aligned with initial. We have equil. condition in A e and B so calculate stresses in red reinf. making equil. around



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PROCEDURE

For design we have m_x^* n_{xy}^* (come from T/N); choose a small portion of structure depending of dim of struct. to analyse we have to repeat calculation for all elem.
→ make equilibrium.
sometimes better to overestimate the effects of actions in order to perform calculus
3/4 times for web
$n°$ of reinf. but we have to consider angle

Try to fix z_1, z_2 → so we obtain from the values of stresses. (σ, τ)

$\sigma_{y1} = \frac{n_y}{z_1}$ $\quad \sigma_{y2} = \frac{m_y}{z_2}$ Actions in I/II layer → thicknesses → stresses in layers 1/2.

$\tau_{xy_1} = \frac{n_{xy_1}}{z_1}$ $\quad \tau_{xy_2} = \frac{n_{xy_2}}{z_2}$

Then we obtain ρ_{y1} and ρ_{y2} and θ_1 θ_2 from
↓
so obtain ammount of steel.

$\tau_{xy} \leq (\rho_y f_{yd} - \sigma_y) \cot\theta$ I know everything apart of ~~about~~ θ, ρ_y

$\tau_{xy} \leq f_{cd_2} \sin\theta \cos\theta$

In our procedure first thing design vertical reinf depends on $V(\tau)$ and normal stress (acting on element)
(acting again).

Repeat procedure for other layers → ρ_{y2} and θ_2 ammount of reinf. needed for action in y direction with that thickness and θ_2 (θ angle for int/ext surface can be ≠)

By $\Delta\rho_1 = \rho_{y1} z_1 \frac{b_w - z_1/2 - c}{b_w - 2c} + \rho_{y2} z_2 \frac{z_2/2 - c}{b_w - 2c}$

$$\Delta\rho_2 = \rho_{y1} z_1 \frac{z_1/2 - c}{b_w - 2c} + \rho_{y2} z_2 \frac{b_w - \frac{z_1}{2} - c}{b_w - 2c}$$

we find real ammount of reinf ρ_{y1} ρ_{y2} are virtual reinf. in y direction
We need to consider real ammount of steel and we can find A_{y1} and A_{y2} → $r = \frac{A_{y1}}{A_{y2}}$

Started with choice of $z_1 z_2$ = level of steel.

Make optimization procedure to minimize value of steel in y direction.
We can modify thickness of layers but $z_1 + z_2 \leq b_w$
$z_1 \geq 2c$
$z_2 \geq 2c$
That's why virtual reinf should be internal real reinforc.
Perform again and again the procedure changing the value of z_1 and z_2 and we obtain for that pattern of load the thickness of element good for us
Then we obtain perfect ammount of steel in y direction
Now we can drawn interaction domain because we know actions (normalized) in terms of m_x, n_{xy} and perfect ammount of steel that is needed for action
We have a point in diagram which has 3 axis 1 n_{xy}
2 m_x
3 ω_y
= z axis

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$z = 0.9\,d = 2.655\,m.$

to consider effect of shear — dist. ext. level of steel

z is always same to have a piece of web involved in action coming from S. Transv. M is due to actions of possible distrib. of load on deck = this possibility has same probability to be along deck
So Transversal action of M is always same, V depends on static scheme. Anyway we choose Δy length along y axis = 0.5 so in this web 6 elements.

Point A/B characteristic values of actions are those: $V_{K_A} = 5 V_{KB}$ (from global analysis) while effect of M is always same.
- Divide by 2 effect of V because two webs.
- Divide by length involved in R action ($z = 2.655\,m$).
- Multiply by height of section = 0.5m

$\cdot 1.5$ = Design value for V and M in A and B.

Now we obtain v_{xy}, m_x
≠ choises
1) $r_y = 1.0 \rightarrow$ for A and B ≠ value of w_y for ≠ V while M = constant (max on pier, min value in midspan)
2) $r_y = 1.5$ not symmetric reinforcement. 2 portion ≠ of steel for each point — ammount of steel
bigger ammount in int. layer and smaller in ext. layer.
Interaction diagram of 1.5 for A A_{s1} A_{s2}
int. layer — ext. layer

longit. reinforc. depends only on shear. $r_x = 1.0$ (generally) → we find ≠ ammount of steel for 2 points (not symm).

Box girder and box shaped pier with skewed diaphragms

Box girder and a box shaped (not diaph. here). Simple to analyze transfer mechanism -> ≠ between of loads at top and bottom flange can be captured by mechanism that goes in same way of diaphragms ~~t~~ = 2 inclined diaph. We can recognize tensed /compressive struts directly.

$\Delta T_i \cdot z_i = M$ -> couple of F C_p T_p -> $C_p \cdot z_p$ = equilibr. with other action. In this way we can find ~~value~~ of T_p

Layout of reinforcement

- Top flange: continuous reinforcem. because of bending $M<0$ at top of pier (comes from longit. analysis).
- Tensile reinforc. ensure this = design shape of reinf. to avoid problem due to discontinuous reinf. in tensile zone

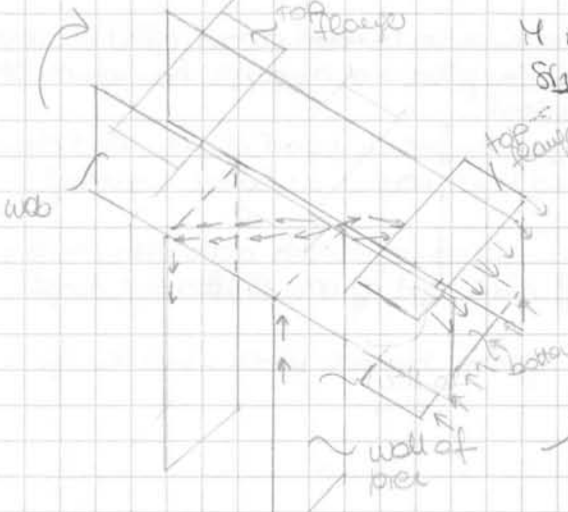
Obviously problem can be in other direction so we need to put other reinforcement. Because side of pier can be compressed / tensed but problem can be in opposite direction.

Best way to solve problem of transfer mechanism of load from deck to pier -> inclined diaph. are very difficult to obtain -> necessary holes for inspection so it isn't so good concreting inclined surfaces

Box girder and box shaped pier with vertical diaphragms

Common solution -> same thickness of pier for vertical diaphragms.

Transfer mechanism of M ~~between~~ between deck / pier is the upload of load not in inclined direction (because diaph aren't skew). So it isn't possible direct transferring of stresses from deck to pier but it's general st. When we construct br. we manage with this problem and put reinf. of in zone imagine direction and deviation of load into ≠ part of section of br. = actions that should deviate into pier have to concentrate firstly in webs and afterwards they have to spread again the walls of the pier + to the webs => strut and tie system.



M transferred from deck to pier

Stress come from top flange and bottom flange

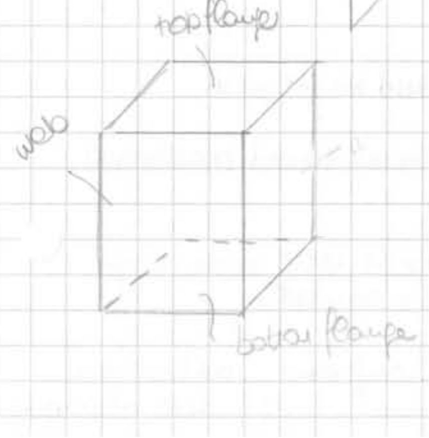
We have stresses on M plane = deviate into vertical plane -> then deviate in vertical plane but ⊥ to plane of web.

At end we have stresses ~~in~~ ~~plane~~ in 2 walls.

On other side they are opposite in sign for equil.

Something in bottom / top flanges that should deviate into webs -> to walls of pier.

Direct transfer isn't possible. Strut is ↯ missing



Top flange connection with webs so we have stresses = load $T_{1/2}$ and $T_1 - \Delta T_1$ -> change of stress is uniform distributed because of connect with web. Simplified assumpt. because distrib. of stresses not uniform.

On web we have distrib. of stresses opposite in sign what acts from top flange to web acts with opposite sign from web to top flange from pretensed prestressing → analyse stresses how prestr. goes to concrete until a certain distance → in which we recognize amount of prestr. transferred to concrete and steel section. While on other side we have ext. loads that create M = if cracked in some parts we have prestr. acting against load but stress calculated in cracked section must have a proper anchorage and lenght of anchorage connected to pull out problem = ensure that the prestr. is adequately anchored and connected to concrete for a lenght that depends on stress acting on structure

Problem of PUSH IN = PRESTRESSING DISPERSION.
Formula regulates according EN 1992 (Part 1.1)
Bond stress = F transmitted to concrete

$f_{bpt} =$
$f_{bpt} = \eta_{p1}\, \eta_{1}\, f_{ctd}$

- f_{ctd} = tensile strenght concrete at time t how pretensed beam ensure this lenght when release cable
- $\eta_{p1} = 3.2$ for 3/7 wires strands depends on type of prestr. used. If we have indented wires we have coeff = 2.7 → bond stress ↓ because ext § of prestr device can give high bond stress. so coeff ↓
- $\eta_{1} = 0.7$ bad condition of bond
  1 good "
  During concreting bottom part of element is good bond condition, while upper bad cond.
  → we have big ≠ in terms of efficiency of bond.
  Concreting is made on depth less of 300 mm = always good bond condition → Find in EC in part for SLS.

TRANSMISSION LENGHT depends of f_{bpt} developed in structure

$$l_{pt} = \alpha_1 \alpha_2 \phi \frac{\sigma_{pm0}}{f_{bpt}}$$

depends directly of σ after release < φ — high σ bigger lenght for transmiss. of load
" of φ, of type of release

α_1 gradual release $= 1$ — α_1 sudden release $= 1.25$ (25% more of l_{pt})

α_2 depends of n° of wires $= 0.19$ (3/7 strands)
$= 0.25$ for ≠ type of strands and n°

This is phenomen is for diffusion and dispersion of load for prestr. of concrete.
If I have a problem of verificat of local stresses at time of release → ensure those high stresses are avoided we use worst situation. →

$l_{pt1} = 0.8\, l_{pt}$ → More stress in an area → verify of local stresses

$l_{pt2} = 1.2\, l_{pt}$ = (20% more) for limit state as V, anchorage when I need stresses but assumed that F give uncertainties → so assume 20% more in order to be in worst conditions. Anchorage should be ensured for a certain lenght = if we increase lenght of transmission zone we reduce possibility.

When we have bond stress at ULS = ≠ calculus to ensure that prestr. is adequately anchored to cls → formula based on f_{ctd} but we consider $\eta_{p2} \neq \eta_{p1}$

$\eta_{p2} = 1.2$ for 3/7 wires stresses

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↓ Bond stresses that it can transfer due pull out problem.

Average value of anchorage length is

400 mm for (a)
↓
because we have triangular section, total distance axis of bearings up to end of box girder
is 800 mm for (b)
710 mm for (c)

$$\text{Resist. action} = \frac{\text{available anchorage length}}{\text{fund transmission length}} \cdot n^\circ \text{ of strands} \cdot \text{fund level of stresses after losses.}$$

$$F_{rd} \approx \left(\frac{400}{830} \cdot 16 + \frac{800}{830} \cdot 27 + \frac{710}{830} \cdot 3\right) \cdot 165.4 = 5359 > F_s$$

↓ tensile F in box section.

FLANGE WEB CONNECTION

Problem connected to shear lag problem of flange / web. When we have box section shear lag problem is present = probl. of transm. of load, of V of web to flange → divide structure into top flange and bottom across web.

- Tensed chord ⊥ to direct of longit stresses

- Compressive struts acts on web but in central part. They should be diffusion in top flange. During diffusion (good because we can divide the stresses along top flange) we need to consider transversal effect of tension so we have tensed chord all over.

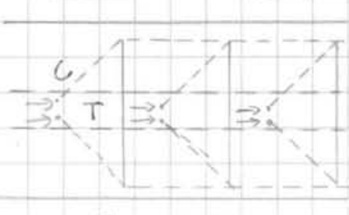
Same mechanism appears into tensed flange all along tensed chord.

Put reinf not only in tensed part because we haven't enough space so reinf. in bottom flange distributed along the section
Then we have a diffus. of stresses along inclined line of ≠ θ of V → we have same problem in terms of tensed chord and compressed chord in tensed region

03/12/08

Simple truss model for connection of flange / web

1° picture effect on top flange of beam. → simply supported beam loaded by a couple of forces.
Having ~~any~~ flange at top / bottom = ~~any~~ problem of diffusion → because we want to ↑ properties of beam especially in terms of M of R action = we want some flange at top and more space for tensile reinforcement at bottom. So we need a double T section (I). Due to this kind of problem → not only M but also V (they are connected).
2° pict. = TRUSS MODEL : compressive / tensile struts
C. struts are closed to truss model and when they come to top flange they can't be spread into top flange.
Model of EC2 → obliged to spread a certain ammount of load into top flange and into bottom flange → Here ≠ behaviour (truss model in which we can choose ammount of stress in ≠ part of section → it's strut & tie model so we can choose the way of flow in T / C. chord → but guarantee equilibrium.
1° Compressive struts angle at top flange = θ_f → what of flange (top) is involved in R action depends on the angle and also stresses in transv. reinforcement.



C. strut in equilibrium with T. stress. They go in top and bottom.
At bottom ≠ spread and θ is bigger.
The spread goes to point chosen = addition of T. stress at bottom and same C. stress at top. This is what happen in web and h_i = height of top flange.

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