Finite Element Analysis of CRCP Slab Track System Designed with Active Crack Control

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Abstract*—Ballastless track is designed for long time period of service up to 60 years or even more. It has main goal to achieve a high performance and less significant maintenance during the service. One development of the standard design, which can be introduced to improve a conventional in-situ casted railway slab track, e.g. continuously reinforced concrete pavement (CRCP), is by implementing active crack control construction type. In this system, a CRCP slab is cut in a spacing interval immediately after it reaches a certain level of sufficient hardening state. This study is conducted to discuss the standard design procedures and the performance of slab track using CRCP as rail supporting structure (e.g. Rheda 2000) based on the long year experience of its implementation in Germany and to study parameters of cut spacing and subgrade bearing capacity of the Rheda-2000 designed with active crack control system. A static Finite Element Analysis (FEA) has been carried out using 3D model in ANSYS to assess the performance of the standard Rheda-2000 designed with active crack control system based on ultimate limit state design criteria. The substructure support is also ranged to investigate the limit performance of the system and to represent different levels of subgrade bearing capacity. The assessment is mainly based on the safety factor and comprises a combination of theoretical, analytical, empirical and FEA methods of ballastless track design procedures. The results demonstrate that there is a critical length of cut spacing of the slab and certain required bearing capacity limit of the substructure to achieve an equilibrium and optimal slab track designed with active crack control system.

Keywords*— *crcp, active crack control, slab track, fea

I. Introduction

The challenges of the increase of traffic, train speed and load and the needs of having long life stability and high reliability of a railway track were the initial background idea of implementing non-ballasted track system. Although ballastless track systems are correlated to relatively higher initial construction cost than that of the ballasted track, these systems feature some major advantages, including: long life cycles, lower in overall cycle cost, low maintenance, and high stability.

The developments and implementations of slab track system have spread all over the world, with different types of applications such as using continuous slab, jointed slab, precast (prefabricated) slab with discrete rail seats or embedded rail systems, which are focused in some case, on tunnel, bridge, or in a high-speed line. In some countries, the ballastless track is also further developed to compromise with soft soil subgrade condition. This study examines the design parameters and performance of ballastless track designed with active crack control system. The long-year-experience in Germany for developing this system is used as the background of discussion in this study.

II. Ballastless Track System in Germany

The first implementation of ballastless track system in Germany was in Hirschaid and the railway station Rheda. The Rheda system was introduced in 1972 and has still indicated a good performance up today after more than 40 years in service and more than 750 MGT of train traffic load. There is no replacement in the fastening system and the rail, which are still from 1972. The maintenance is only done for rail grinding and beside of that no other maintenance. In this system, the sleepers are mounted into an in-situ casted continuously reinforced concrete slab (see Fig. 1). From 1972, the Rheda classic system was further developed in some stages [1]. Other ballastless track systems using asphalt and unbound base have also been developed in Germany, such as SATO-FFYS system (1995), Walter system (1995), GETRAC system (1995)[4][5].

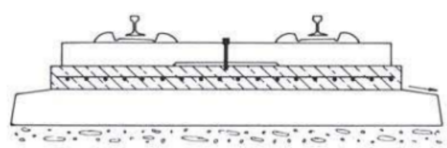

Fig. 1. The sketch of sleeper panel mounted in a continuously reinforced concrete pavement (CRCP)

One example of best-known and well-innovated generation of Rheda system is Rheda-2000, which was developed in 1998 [2][3], as depicted in the Fig. 2. The superstructure parts of Rheda-2000 may consist of rail profile 60E1/E2 (formerly named as UIC60), resilient fastening system (e.g. IOARV 300), prefabricated reinforced twin block sleeper B355.3, and continuously reinforcement concrete pavement (CRCP) slab of 240 mm thick. The concrete slab is in-situ casted with the twin block sleepers to have a monolithic and solid structure. The design of the concrete slab generally follows the German Highway Construction Regulation for Concrete Pavement [6]. It requires a minimum diameter of the steel reinforcement bars (rebars) of the slab is 20 mm and the total amount of the rebars area is 0.8 - 0.9% of the slab cross section area [4][5][7]. The rebars are positioned near to the middle of cross section of the slab. In the design, the crack width is limited up to maximum 0.5 mm.

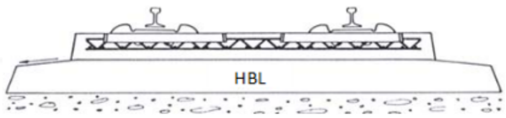

Fig. 2. The illustration of Rheda-2000 slab track system

This system is supported with hydraulically bonded layer (HBL) like concrete treated base (CTB), and also with frost blanket/protection layer (FBL/FPL) below the HBL. The design of HBL generally follows the German Highway Construction Regulation for Concrete Pavement (ZTV Beton-StB 07/2013) [6]. In the standard design of Rheda-2000, HBL should have minimum thickness of 300 mm and elastic modulus of 5,000 - 10,000 MPa.

According to German regulation (Deutsche Bahn: Ril 836)[8], the FPL has to have minimum thickness of 500 mm and modulus deformation Ev_2 of 120 MPa on the top layer of FPL. Ev_2 value is widely used in Europe as a practically field measurement method to assess the bearing capacity of embankment or subgrade, subsoil, or unbound granular material (UGM). It is obtained from the second loading of plate bearing test according to DIN 18134 [9]. The German guideline ZTV E-StB 09/2012 [10] regulates about the earthwork. It mentions that the Ev_2 value of the subgrade should be not less than 45 MPa in the design state, but 60 MPa is recommended in the application state[4][5].

III. Design Procedure of Ballastless Slab Track

A. *General Overview of Conventional Design Procedure*

One of the theoretical method of designing ballastless track is the combination of Zimmermann and Westergaard methods as proposed by Eisenmann (2000)[11]. Developments of these methods, for instance the ones suggested by Freudenstein, et. al (2015)[12] and Prakoso (2017)[14]. It is done in two steps. This is possible because the stiffness of the rail and rail support is much lower than the stiffness of the pavement system on standard substructure (except soft soil). The first step is the calculation of the stresses on the rail and rail seat forces (or rail seat loads) by using Zimmermann method. This method assumes a transformation of a beam to a continuous elastic support. In the ballastless track application, that is the transformation of the discrete rail pad supports. The results of the rail seat forces in each position of the rail-pad from the first step are then used in the second step as the discrete loads of the concrete slab. The calculation of the decisive stresses in the concrete slab can be done by comparing the calculated results of Zimmermann and Westergaard methods, which is based on assumption of a beam (Zimmermann) and a slab (Westergaard) on continuous soil support. And then the stresses on the substructure layers until the soil surface can be obtained by using the approaches, e.g. of that by Boussinesq in combination with Odemark's half-space theories.

This method considers a static analysis. Therefore, in the design, to contemplate the quasi-static and dynamic impacts of the running train, a dynamic amplification (or multiplication) factor (DAF/DMF) is employed. DAF is obtained based on the empirical and statistical data from measurements, by considering track quality level, train speed and safety factor. One example of widely used DAF formulation is the approach proposed by Eisenmann (1972) [11][13], as follow:

$$\varphi = 1 + \delta \eta . t \qquad (1)$$

where: δ is the track quality factor, η is the train speed factor, and t is coefficient of variation based on upper confident limit. The illustration of the design procedure of ballastless track system can be seen in the Fig. 5 [14].

These general approaches are extensively used in the conventional design method of ballastless track. Yet, because of the steps are separated, hence the first calculation using Zimmerman takes into account the total stiffness. The resulted of rail seat forces are estimated in this way, with an assumption for the calculation that the bottom of the rail pads are with fixed support, so there is no additional deflection by the pavement/concrete slab. This does not reflect perfectly to the real situation, that the rail, rail pad, sleeper block, concrete slab and substructure are working together.

The deflection within the concrete slab is considered very small in the standard design and non-floating slab, thus this approximation is quite acceptable. However, if the design has to compromise with a weak soil condition or a floating slab system, in which the deflection on the slab might be higher, this is the limitation of using this approach. In addition, these static approaches are not able to be used in the investigation of the track-soil dynamic interaction due to the absence of the structure's mass and dynamic formulation.

B. *CRCP Design Consideration*

In a CRCP design brought from the long year experience of rigid pavement design for a highway, the major function of the continuous reinforcement steel bars (rebars) is to hold tightly together the transverse cracks which occur on the concrete. To protect the concrete against spalling and water penetration, the allowable crack width is limited to maximum 0.5 mm. Based on the experience on highway applications, the major transverse cracks in CRCP occur normally at the interval of 1.1 - 2.4 m [15]. Initial random cracks are naturally formed by critical stresses generated by temperature drop and drying shrinkage of concrete, and 90% occur within one month after construction [16].

In the Rheda-2000 application, it is often found that small cracks with width of 0.1 - 0.2 mm occurs in the connection between the sides of the sleeper blocks and concrete slab in some sleepers along the slab after some time due shrinkage, temperature and traffic loading. These cracks appear because twin block sleeper is prefabricated meanwhile the slab concrete is in-situ casted.

Using the theoretical and analytical methods of ballastless track design as previously described, the conventional CRCP slab is frequently assumed as a slab with infinite length. One approach to incorporate the formation of the random cracks in the infinite concrete slab is by considering the impact of supplementary flexural stresses within the concrete, which may cause natural random cracks due to temperature changes.

C. Active Crack Control System

A general way, which can be introduced to control the crack in CRCP pavement is by cutting the slab concrete short after it reaches a certain level in the early hardening state. The cuts may be located in the interval where the major cracks are expected to form. This method can be named as active crack control (ACC) of CRCP. The cutting process is done by using saw cut machine. An example of application of ACC on CRCP slab track in Germany can be seen in the Fig. 3.

One important thing, which should be carefully considered for implementing this method is a proper timing for cutting the slab. It should be guaranteed that the cutting is done before the formation of natural random cracks as well as small cracks between sleeper and CRCP concrete appears. If random cracking is identified as a dangerous risk, the main function of ACC is to reduce the occurrence of excessive random crack formation. Since those random cracks has potential to decrease the performance of CRCP, ACC will not work properly when the random cracks occur before the cutting. Therefore, this should be avoided.

Fig. 3. Example of application of ACC on CRCP in Germany

D. Curling Stresses on the Concrete Slab due to Temperature Changes

The calculation of the curling stresses generated due to temperature changes of full restraint concrete slab may follow the approach suggested e.g. by Westergaard-Bradbury (1938) and Eisenmann (1979), as it is shown in this formula [17][18]:

$$\sigma_w = \frac{1}{1-\mu} \cdot \frac{\alpha . E . \Delta t}{2} . h \tag{2}$$

Eisenmann (1979)[19] studied the warping stresses at the bottom of a simply supported beam due to positive temperature gradient of thermal load and figured out a critical length *(L_{crit})*. The longer the slab the higher warping stress up to this L_{crit}, but this stress then remains almost the same above this L_{crit} as formulated below[17][18][20]. Due to dimension/shape factor of the slab, the critical length may be reduced into $0.9L_{crit}$ for a quadratic slab, where $0.8 \leq Length/Width \leq 1.2$ and slab's length (or width respectively) $< 0.9L_{crit}$ [18].

$$L_{crit} - h . \sqrt{\frac{4\alpha E\, \Delta t}{5.(1-\mu).\gamma}} \tag{3}$$

$$\sigma_w{}' = (1.2)\sigma_w \qquad if\ L > 0.9L_{crit} \tag{4}$$

$$\sigma_w{}'' = \sigma_w \left[\frac{L}{L_{cr}}\right]^2 \qquad if\ L \leq 0.9L_{crit} \tag{5}$$

Where: Δt is the temperature gradient, α is the thermal expansion coefficient, μ is the Poisson's ratio, E is the Young's modulus, h is the thickness and γ is the dead load per unit length of concrete.

Eisenmann and Leykauf (1990) investigated further the effect of thickness, joint spacing and support condition [21]. Regarding the positive temperature gradient, an uplift deflection occurs. This uplift leads to a contact loss between the slab and the subgrade. Hence, slab is only supported by its ends through a support length of [17]:

$$L' = L - 3\sqrt{\frac{h}{k.\Delta t}}, \tag{6}$$

where k is the modulus subgrade reaction (N/mm^3).

The negative temperature gradient is the source of concave curling on concrete slab. The calculation of this stress follows the Equation (2) above. The critical length for negative temperature gradient is given by this formula [18]:

$$L_{crit(-)} = h . \sqrt{\frac{2.\alpha.E.\Delta t}{3.(1-\mu).\gamma}} \tag{7}$$

From the formulations above, the impacts of thermal load, joint spacing (slab length) and subgrade's bearing capacity can be then included in the design calculation. For the design of a continuous slab with random/free crack, an assumption of slab length below the critical length may be considered. Meanwhile, the impact of the warping stresses due to heating on the slab surface reduces if the slab is built with active crack control with longitudinal joint spacing also below the critical length.

In the active crack control design of CRCP, the percentage of rebars may be reduced in comparison with that one in examples with the random crack design. Reduction of the rebars amount should carefully take into account the length of unit slab (interval of cutting or longitudinal joint spacing). The length selection of the unit slab is related to the impact of temperature changes within the slab. This should consider the resulted decisive tensile stress, by comparing the stresses caused by slab warping due to surface heating and those ones caused by shrinkage and slab curling due to cooling.

E. Ultimate Limit State Design

An ultimate limit state method in structural engineering takes into account a condition of a structure until it reaches the boundaries of the design criteria. This method may include the combination of analytical, empirical methods as well as design corrections and/or safety factors, which may come from the judgment of the engineers based on their experience.

In the railway track design, the structural limit state design criteria is mainly pointed to the limit criteria:

- of strength of the rail component against rail fatigue/fracture (or buckling),
- of the concrete slab against flexural and fatigue damages; and
- of maximum allowable stresses or deflection of the substructure and subsoil layers against excessive settlement and plastic deformation.

In this study, the limit criteria of the rail is defined by considering the maximum allowable oscillating stress due to impact of corrosion on the rail and temperature difference. A residual stress is generated during the differential cooling of rail head, web and foot and rail straightening procedures during the production. Usually, to cover this, a constant value of 80 MPa is used and the stresses caused by temperature difference are also taken into account. Thus, the constant minimum stress σ_u by temperature change can be determined by these formulas [18]:

$$\sigma_u = \sigma_T + 80 = E.\alpha.\Delta T + 80 \text{ [MPa], for steel rail:}$$

$$\sigma_u = 2.52\Delta T + 80 \qquad (8)$$

Where: ΔT is the temperature changes. The allowable oscillating stress σ_d on the rail is determined by using this chart on Fig. 4 or a proximally using linear regression Equation (9) and (10):

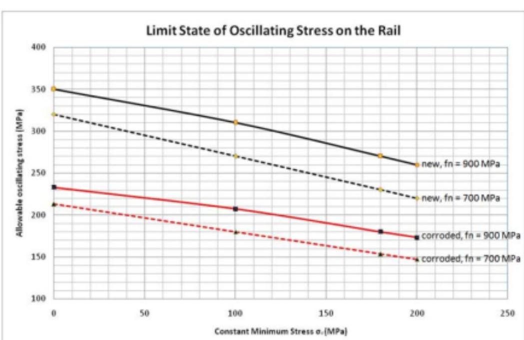

Fig. 4. Limit state criteria of oscillating stress on the rail (after [18])

Allowable oscillating stress on the rail [13]:

- for a new rail:

$$\sigma_d = -0.48\sigma_u + 0.19f_n + 185.85 \qquad (9)$$

- and for a corroded rail:

$$\sigma_d = -0.32\sigma_u + 0.13f_n + 123.98 \qquad (10)$$

where: f_n is the nominal strength of the steel rail (MPa).

For the concrete track components, such as CRCP slab and CTB layer, the limit state criteria of stress can be estimated by using the Smith's approach. This approach superposes the loading cases as a combination of traffic and temperature loads by taking into account the bending strength of concrete under cyclic loading as it is formulated in this equation:

$$\sigma_{Qmax} = \beta_{BZ} \cdot \left[(\log n \quad 2)\left(\frac{0.0872\sigma_W}{\beta_{BZ}} \quad 0.07\right) + 0.8\right] \quad \sigma_t \qquad (11)$$

where: β_{BZ} is the bending tensile strength of the concrete, σ_t is the constant stresses for temperature change ΔT or temperature gradient Δt (see Eq. 2, 4, 5) and n is the number of load cycles $\leq 2\text{x}10^6$ based on laboratory fatigue test. In the design and field application, load cycles during the presence of temperature loading $n_{\Delta t}$ value can be taken as 5% (determined for temperature regime in Germany) of the expected load cycles during the service life of the track [18].

The distributed stresses from the traffic load and superstructure part should be reduced to the subgrade layer under the limit of its bearing capacity. The criteria in the static design is mainly seen at the maximum stress at the soil subgrade's surface to guarantee certain safe limit against disproportionate plastic deformation and settlement after cyclic loading during the service.

There are many advanced approaches developed especially in the field of geotechnical engineering to take into account soil subgrade's parameters more in detail. Yet, a conventional approach, e.g. fatigue model after Heukelom and Klomp (1962), is still frequently used to approximate the mechanistic failure in the substructure and soil layers under a cyclic loading. Although this mechanistic failure model is simple, but it is quite useful to make general consideration in the practical application in railway track design. This approach gives suggestion of the maximum allowable stress limit ($\sigma_{z,allow}$) of the substructure layer by only considering the property of dynamic modulus of the material (E_{dyn}) and the number of load cycles (n), as can be seen here:

$$\sigma_{z,allow} = \frac{0.006E_{dyn}}{1+0.7\log(n)} \qquad (12)$$

The summary of analytical design procedure of railway track using ultimate limit state criteria and safety factor assessment can be seen in the Fig. 5 (after [14]).

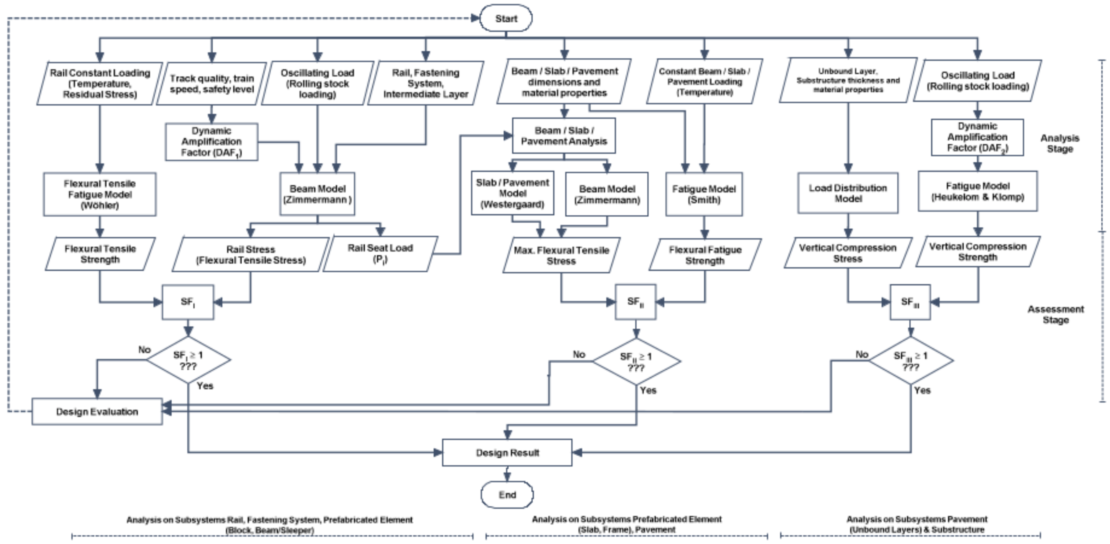

Fig. 5. Schematic view of the analytical design procedure (after [14])

IV. Finite Element Analysis (FEA)

In this study, by employing the Finite Element Analysis (FEA) in software ANSYS, the performance of Rheda-2000 is assessed by looking at the condition of random cracks in a semi-infinite longitudinal length of CRCP slab and active controlled cracks performed by cutting of the concrete slab in some intervals from 1.95 to 12.35 m.

A detailed 3D FE-model is built in ANSYS to analyze the performance of Rheda-2000 built with ACC as can be seen in the Fig. 6. The solid parts of the structure is idealized using ANSYS's solid element and linear isotropic material properties, contacts among them using contact element, rebars using link element and for soil using combination of solid and surf elements with homogeneous modulus elasticity as well as elastic foundation stiffness (EFS) property[22].

The verification is done for the basic model to confirm the validity and reliability of the model in comparison with the manual calculations of Zimmermann and Westergard methods. The load applied in the model is Load Model 71 (LM71) for the static analysis. LM71 was commonly used for traffic load design for a bridge, and then it is implemented in the ballastless track design. As suggested by Lechner (2008)[5], a reduction of 80% can be done if the design only takes into account the high speed trains and factor 1.2 and 0.8 is taken at the field- and in-side rail within a curve respectively.

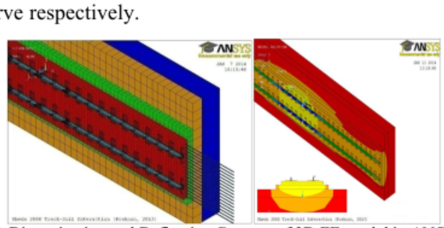

Fig. 6. Discretization and Deflection Contour of 3D FE-model in ANSYS

The dimensions and material properties of rail components set for the main analysis:

- × Applied load: the theoretical Load Model 71 (LM71), and taking into account a dynamic amplification factor (*DAF*) of 1.4
- × Continuously welded rail profile 60E1 supported with a fastening system with static stiffness of 22.5 kN/mm
- × Twin block monolith concrete sleeper with Young's modulus of elasticity (E) = 34000 MPa, Poisson's ratio (μ) = 0.2 and sleeper spacing 0.65 m
- × Reinforcement bars (rebars) of CRCP, with E = $2.1\text{x}10^5$ MPa, $\mu = 0.3$, $Ø = 20$ mm
- × CRCP slab with thickness (h) = 240 mm, E = 34000 MPa, μ = 0.2, bending tensile strength β_{BZ} = 6 MPa, built in 4 model variations:
  - ✓ semi-infinite continuous slab **Model I** without cutting, and total amount of rebars area (ρ) of 0.8%
  - ✓ active crack control **Model II** with maximum crack width (*Cw*) of 0.5 mm, active crack control spacing (*Cs*) of 1.95, 3.25, 4.55, 5.85, 7.15 m, ρ = 0.5% and with an assumption of **full bond** interface between concrete and rebars within the dummy joints
  - ✓ active crack control **Model III,** like Model II but with *Cs* = 5.85, 7.15 and 8.45 m, and ρ = 0.6%
  - ✓ active crack control **Model IV,** like Model II but with *Cs* = 9.75, 11.05 and 12.35 m, and ρ = 0.7%
- × Hydraulically bonded layer (HBL) by employing CTB layer with h = 300 mm, E = 5000 MPa, μ = 0.2, β_{BZ} = 1.6 MPa and **full bond contact** between HBL and CRCP and HBL and substructure layer
- × The conversion of field parameter of modulus deformation (Ev_2 in MPa) in the standard design into EFS or modulus subgrade reaction k (MPa/mm) parameter in FE-model can be done by utilizing the approach by Timoshenko and Goodier (1951)[23], which is based on Boussinesq theory:

$$k = \frac{2E}{\pi . r(1-\mu^2)} \tag{13}$$

where E = modulus of elasticity, r = radius of plate bearing test (150 mm).

The modulus of elasticity (E) in that formula brought by Boussinesq theory and modulus deformation (Ev_2) obtained from plate bearing test (PBT) are actually not the same. The reason is because of the soil's stress-strain elastoplastic behaviour obtained from PBT. But if a small load is applied on a stiff and elastic behaviour material, those moduli can be assumed identical [24].

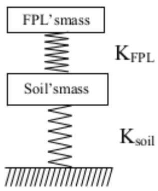

Fig. 7. Substructure model

As illustrated in the Fig. 7, the total value of spring constant (k_{tot}) is a combination of a FPL and soil springs, which are in series. This k_{tot} can be defined by using this formula:

$$\frac{1}{k_{tot}} = \frac{1}{k_{FPL}} + \frac{1}{k_{soil}} \tag{14}$$

Due to the simplification in the FE-model that the FPL and soil behaviours are assumed linear elastic, hence the EFS value is taken by using the conversion formulas (13) and (14). The value of Ev_2 = 120 MPa and μ = 0.25 of FPL gives k_{FPL} = 0.54 N/mm^3; and Ev_2 = 60 MPa and μ = 0.25 of subgrade results k_{soil} = 0.27 N/mm^3 respectively. The combination of those k values of two layers is a proximally k_{tot} = 0.18 N/mm^3. The reduction of Ev_2 = 45 MPa of the soil gives the k_{tot} = 0.15 N/mm^3. The other variations of k_{tot}= 0.5, 0.3, 0.2, 0.18, 0.15, 0.1, 0.05 and 0.02 N/mm^3 of EFS for the surf element and $E \approx$ 110, 66, 44, 49, 33, 22, 11 and 4.4 MPa respectively for the solid element are simulated for the FE-analysis.

V. Results and Discussion

The values resulted from FE-analysis are compared to their ultimate state limit criteria of each track's component, which is calculated using the approaches above. For assessment of the result, the safety factor (SF) values are used in this study.

$$SF = \frac{\text{Ultimate Limit Criteria}}{\text{Value from FE Analysis}} \tag{15}$$

In the design application, the SF criteria is relative in its scales, and its interpretation is subjectively based on the experience and the judgment of the engineers. In this study, the theoretical criteria is used that if the $SF < 1$ can be considered to be risky (unsafe) design, $SF = 1$ is the critical limit design and $SF > 1$ is in a safe side of design.

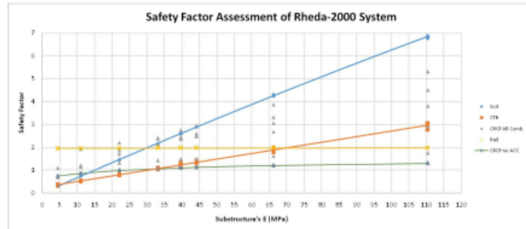

Fig. 8. Assessment of Rheda-2000 Components based on Safety Factor

Fig. 8 and Fig. 9 show the overall results from FE-simulation with total of 80 combinations of Model I to IV variations. Fig. 8 presents that the safety factor (SF) values of rail, HBL/CTB and substructure demonstrate almost linear correlation to the variation of the substructure bearing capacity. Yet, the SF values of the CRCP are vary in response to the variation of the substructure bearing capacity and longitudinal joint interval (cut spacing).

The critical $SF = 1$ of the system is reached in the substructure bearing capacity of 33 MPa. This is decisive especially for the concrete elements (CRCP and CTB). From the chart above, it can be seen that the substructure stiffness of 45 MPa already provides sufficient bearing capacity for a moderate safety factor. In the standard design of Rheda-2000 without active crack control (ACC), the equilibrium structure design is achieved in the condition of substructure bearing capacity of about 60 MPa, which is more recommended in the application state. This gives optimum rail deflection and sufficient safety factor for all track components, but not too significant to the CRCP as it has the lowest SF. But it should be kept in mind that this assessment based on the linear estimation using ultimate limit criteria assuming the condition at the end of track service period. CRCP is considered to be very expensive in the construction, therefore, designing CRCP for ballastless track with sufficient SF is also to avoid an overdesigned construction.

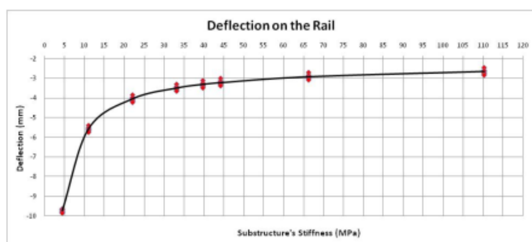

Fig. 9. Rail deflection of Rheda-2000 with applied Load Model 71

The resulted deflection of rail more than 3 mm is quite high for a high speed train application (see Fig. 9). In this analysis, it is resulted due to the high applied static load of LM71 and low bearing capacity of subgrade. In addition, the stiffness of 22.5 kN/mm of the rail pad used in the model gives higher deflection, but it also gives an advantage of wider distribution of the load to more rail support points. An example of manual calculation using Zimmermann method represents that there are 20% reduction of rail seat force and 20% increase of deflection by using rail pad with stiffness of 22.5 kN/mm in comparison with that one on the rail pad with stiffness of 40 kN/mm [1].

There is a critical length of cut spacing, where the stress remains almost the same although the cut spacing is increased. In the condition of moderate until high level of substructure bearing capacity, the critical length is found at 7.15 m and then for the cutting lengths more than that, SF values begin to continue almost similar (see Fig. 11). This occurs because slab is continuously supported by a sufficient stiffness of substructure.

If the bearing capacity of the substructure is low, then it needs longer cut spacing to reduce the stress. However, if the slab is increased until to become too long, then in the one hand the bending stress of the concrete will be more dominant and also increased. In the other hand, a longer slab will increase the warping stress within the concrete as well. This impact is even more obvious if the slab is laid on the substructure with low bearing capacity. Therefore, this critical cut spacing (unit slab length) is important in the design to achieve an optimal structure.

For *SF* calculation, the allowable limit stress is also influenced by the warping stress. As discussed before and it is shown in the Equation (3), (4) and (5) that there is critical limit for warping stress as well, which is about 7.5 m for the given data of Rheda-2000 above and considering the heating on the concrete slab. Therefore, the *SF* on the CRCP built with active crack control is also influenced by this critical warping stress as can be seen in this following figure:

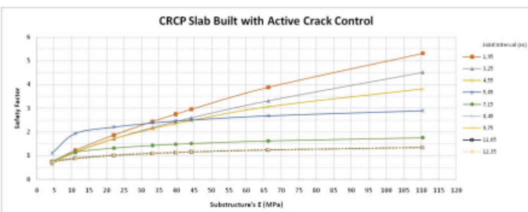

Fig. 10. Safety Factor of CRCP slab built with active crack control (ACC)

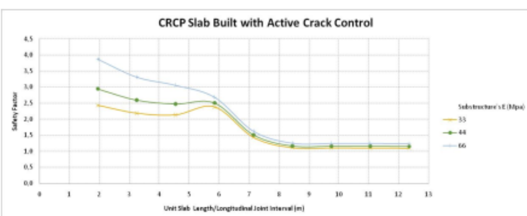

Fig. 11. Impact of the Length of Cut Spacing to Safety Factor in the Substructure Stiffness of 33-66 MPa

Since the longer the slab length (cut spacing) the lower the *SF* value, therefore the design of minimum cut spacing has to consider the slab width in the transverse direction. It is suggested that the minimum longitudinal cut spacing should be not less than the slab width. From Fig. 10 and Fig. 11, it is indicated that proper selection of cut spacing gives benefit to the increase of safety factor of CRCP slab. If the suggested value of minimum substructure bearing capacity of 60 MPa is considered to gives sufficient *SF* for CTB, then the cut spacing of 5.85 m (less than critical) provides the greatest *SF* of 2.68 for CRCP as it is shown in the Table 1.

TABLE I. OPTIMIZATION OF RHEDA-2000 BUILT WITH ACC BASED ON THE SAFETY FACTOR VALUE

E Substructure (MPa)	ACC Spacing (m)	SF			
		Rail	*CRCP*	*CTB*	*Soil*
33	5.85	1,98	2,38	1,09	2,17
	7.15 (crit)	1,96	1,43	1,08	2,17
44	5.85	1,98	2,51	1,36	2,92
	7.15 (crit)	1,97	1,51	1,34	2,92
66	**5.85**	**1,98**	**2,68**	**1,91**	**4,30**
	7.15 (crit)	1,96	1,62	1,89	4,30

Increasing the thickness of the CRCP might be an alternative solution for designing ballastless track with ACC in the lower stiffness of substructure. The bending tensile stress is normally reduced for a thicker slab. However, there is almost no significant different of bending stress although the thickness is increased, if the cut spacing is also increased as it is depicted from Fig. 12. Therefore, thickness and cut spacing parameters play an important role to achieve an equilibrium design of CRCP built with ACC.

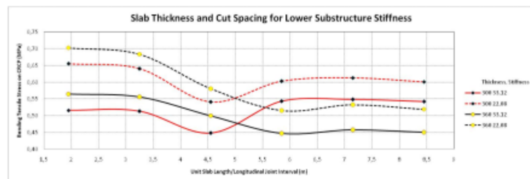

Fig. 12. Bending tensile stress in CRCP with variation of slab thickness and substructure stiffness

The traditional assumption is that increasing the slab thickness gives the best solution if the substructure bearing capacity is lower than the standard. However, the *SF* values resulted from FE-Analysis in the Fig. 13 exhibit that the optimal *SF* is reached not in the thicker concrete slab if the cut spacing is less than the critical length. This indicates that the design should also take into account the influence of slab dimension (length and thickness) and temperature changes to the impact of flexural strength of slab. This is more decisive if the slab track is subjected to compromise the condition where extreme temperature changes occur and low bearing capacity of soil exists.

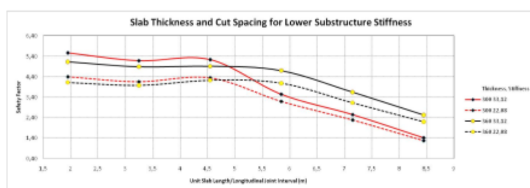

Fig. 13. Safety Factor of CRCP with variation of slab thickness and substructure stiffness

The benefits of implementing active crack control system on concrete slab track design then can be summarized as follow:

1. proper selection of cut spacing provides an ability to increase durability of the concrete slab performance due to the reduction of the bending stress on the concrete

2. more economical structure due to the possibility to reduce the amount of steel reinforcement

3. more equilibrium and compact structural track system can be achieved
4. able to compromise problem with a condition of extreme temperature changes

For further studies, some important design considerations in the design of railway slab track using CRCP built with ACC are:

1. a proper timing of the cutting process. This should avoid the random cracks form before the cutting process is done.
2. the effects of debonding contact between concrete slabs should be also taken into account
3. the impacts of load transfer efficiency within the dummy joints should be considered
4. a proper selection of thickness and cut spacing of concrete slab in the design
5. more detailed analysis in the soft soil condition.

VI. Conclusion and Recommendation

The static FEM analysis and simulation have been carried out using 3D FE-model in ANSYS to assess the standard Rheda-2000 system and a proposed design built with active crack control (ACC). It is shown that a low bearing capacity of soil below the standard impairs the equilibrium of the structure. Therefore, in the practice, a sufficient stiffness of 60 MPa of substructure is suggested, otherwise for an extraordinary condition of subground e.g. soft soil, a deeper analysis especially in the geotechnical aspect is absolutely required.

The cut spacing, thickness of the concrete slab, bearing capacity of substructure and temperature parameters play very important role in the design of slab track constructed with ACC. In addition, modification of the design should be carried out by carefully considering the superstructure, trackbed supporting layers and soil parts to achieve an optimal and equilibrium structure.

It is found that there is critical cut spacing, which may provide an optimal improvement to the standard slab track using CRCP and designed with active crack control system. It is also figured out that implementing ACC in slab track construction is able to optimize the performance of the conventional slab track.

This study has presented general overview of an idea of theoretical implementation of ACC in CRCP acting as a rail supporting structure in ballastless track systems. However, there is still a need to investigate this topic in more detail regarding debonding effect, impact of joint efficiency, the practical method of cutting in the realization, deeper analysis in temperature impact on concrete, and advanced geotechnical consideration of soft soil. Finally, it is highly suggested that this study is proceed with real model in the laboratory and also supported with experimental and measurement data.

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